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Assessment of the Structural Integrity of the People Center, Building 16, Chanute Air Force Base, IL

by
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USA-CERL inspected the building and identified specific problem areas. In general, the major problem is the flat roof system, which has no provision for drainage and is stressed due to the overload from a second layer of roofing installed over the original. This condition allows water to pond on the roof, creating an additional dead load on the structure. Water ponding had been identified as the probable cause for collapse of the auditorium roof. Other problems noted with the building are its age, termite damage, and cracks in the first-floor ceiling due to an imbalance of dead loads on the second floor. It was not within USA-CERL's scope of work to determine the extent of termite damage.

Based on the inspection and stress analysis, Building 16 was determined to be safe for occupancy over the near term except for the occasional falling ceiling tiles in some rooms. However, a replacement/repair cost analysis showed that the expense for replacing the entire roof probably would not be justified in light of the building's age and other problems. USA-CERL has recommended phasing out Building 16 over the next few years.

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FOREWORD

This investigation was performed for the Chanute Technical Training Center, Chanute Air Force Base (AFB) IL, by the Engineering and Materials Division (EM), U.S. Army Construction Engineering Research Laboratory (USA-CERL) under MIPR No. DE87-285 during June through July 1987.

The authors wish to acknowledge the support of Virlon Suits, David Peters, and shop engineering personnel of the 3345th Civil Engineering Squadron at Chanute. Appreciation also is expressed to William Gordon and Donna Schaal, USA-CERL-EM, for their contributions. Their support was key to the timely accomplishment of this investigation. Dr. Robert Quattrone is Chief of EM. The USA-CERL technical editor was Dana Finney, Information Management Office.

COL Carl O. Magnell is Commander and Director of USA-CERL, and Dr. L. R. Shaffer is Technical Director. GEN Peter D. Robinson is Commander of Chanute AFB.

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ASSESSMENT OF THE STRUCTURAL INTEGRITY OF THE PEOPLE CENTER, BUILDING 16, CHANUTE AIR FORCE BASE, IL

1 INTRODUCTION

Background

A catastrophic collapse of the auditorium roof of the People Center, Building 16, Chanute Air Force Base (AFB), IL, occurred on 14 April 1987. An initial analysis of the failure indicated several contributing factors. The auditorium was a single-story structure with standard wooden Pratt trusses spanning 57 ft 6 in.* The auditorium space had been converted into offices, and suspended ceilings, lighting fixtures, heating, ventilating and air-conditioning (HVAC) fixtures, communications equipment, and other retrofits had been added under the trusses. In addition, the roof, originally pitched and constructed with a slope of approximately 6 in. over 28 ft 9 in., had been repaired with the new roof system placed over the old system, adding about 6 lb/sq ft to the roof dead load. Truss inspections had been performed regularly and needed repairs completed. The most recent truss inspection occurred a week before the failure and the discrepancies had not yet been corrected.

Due to the factors cited above, it is likely that the trusses deflected, the pitch became a depression, and rain and melting snow ponded on the roof. Ponding was observed the morning of the failure.

The day after the roof collapse, the Chanute Base Engineer asked the U.S. Army Construction Engineering Research Laboratory (USA-CERL) to inspect Building 16 and ascertain the safety of reoccupying the portion of the building that remained standing after the failure of the auditorium space. It was determined that the remaining structure was safe for occupancy, but significant ponding problems were noted on the remaining two-story roof area of Building 16 (Figure 1*). Concern for this condition prompted the Chanute Base Engineer to commission a structural inspection and evaluation of the remainder of the facility. USA-CERL was again asked to perform this investigation.

Objective

The objective of this investigation was to evaluate the structural condition of the portion of Building 16 that remained standing after failure of the auditorium roof to (1) determine the ability of members and connections to resist applied loads, (2) identify specific problem areas, and (3) recommend corrective action as required.

Approach

The original construction drawings for Building 16 were reviewed to identify critical members and framing details. Drawings associated with the various upgrade to the

*Metric conversions are given on p 22.
*Figures and tables are located at the end of the text beginning on p 24.

building were also consulted to verify changes to the structure. The roof surface was surveyed to determine the location of sections at which the maximum deflections existed. These roof sections and other selected areas were then inspected visually to assess the condition of the structural members/connections and the structural environment. Knowledge of the actual structural conditions facilitated analysis of typical structural sections under critical load conditions due to dead load, ponding, wind and snow loads, and realistic live loads. The effects of these loads on critical members and connections were evaluated. Based on the results of the visual inspection and structural analyses, the building's structural integrity was assessed.

Mode of Technology Transfer

The techniques employed and the information gathered in this investigation will be integrated into the draft Engineering Manual *Structural Inspection of Wood and Masonry Buildings*.

2 STRUCTURAL INSPECTION

The original construction drawings and successive drawings developed for maintenance and upgrade of the building were reviewed to identify critical members, framing details, and alterations to the building. The original carpentry specifications were also examined. Information regarding loads which the structure was originally designed to support was not provided on the drawings or in the specifications.

Description of the Structure

Building 16 was constructed in 1951 and is a vintage flat-roof, two-story, L-shaped wood structure of approximately 40,050 sq ft with a single-story 4850-sq ft auditorium attached. It was originally designed to serve as an All-Purpose Academic Building with several large classrooms typically 22 ft 8 in. by 46 ft 6 in. Partition walls were added prior to 1971 but no drawings exist documenting this alteration to the structure. The building now serves as the "People Center," housing offices for in-and outprocessing of civilian and military personnel.

The roof of the auditorium collapsed on 14 April 1987 and this portion of the building was removed from the site. A one-story brick addition attached to the north wing of the original two-story structure was not evaluated. The remainder of the original two-story structure consists of two wings intersecting at right angles (Figures 2 through 5). First-floor, second-floor and roof plans for the structure are shown respectively in Figures 6 through 8 and a typical cross section is shown in Figure 9.

Review of Drawings

The gravity load resisting system for Building 16 is standard wood construction: wood sheathing spans over wooden roof and floor joists supported by wood stud load bearing walls that carry the loads to a concrete block foundation wall and a reinforced concrete strip footing. The roof system is constructed of 1 in. by 6 in. decking* over 2 in. by 14 in. joists at 16 in. on-center spanning 23 ft 6 in. over the classrooms, and 2 in. by 8 in. joists at 16 in. on-center spanning 10 ft over the hall corridors. The second floor system consists of 1 in. by 6 in. diagonal decking over 2 in. by 14 in. joists at 12 in. on-center, and 2 in. by 8 in. joists at 12 in. on-center over the classrooms and hall corridors, respectively. The first-floor system is a 4-in. reinforced concrete slab reinforced with 6 in. by 6 in. 10-gauge reinforcing wire mesh on 6 in. of gravel fill.

The lateral load resisting system of the building consists of diagonal sheathing over wood studs on the exterior walls. Anchor bolts, 5/8 in. in diameter at 4 ft on-center, are provided at the intersection of the first-floor sill plate and the concrete block foundation wall.

The original construction drawings for Building 16 indicate that the exterior and interior load bearing walls are supported on strip footings at an elevation about 5 ft below finished grade. The exterior strip footings are 20 in. wide and 8 in. deep with three 0.5-in. diameter reinforcing bars. A 12-in. concrete block foundation wall is constructed

*Unless otherwise noted, all lumber dimensions in the text of this report are nominal dimensions.

on top of the strip foot to the bottom of the first-floor reinforced concrete slab on grade. The interior strip footings are 16 in. wide and 8 in. deep with three 0.5-in.-diameter reinforcing bars. An 8-in. concrete block foundation wall is constructed on top of the interior strip footing and extends to the bottom of the first floor reinforced concrete slab on grade. In addition, about 4 ft inside the exterior wall, there is another strip footing and concrete block foundation wall which forms the pipe tunnel for the hot-water heating system servicing the building.

Review of Carpentry Specifications

The material properties stated in the carpentry specifications are as listed in Table 1. The lumber was to be air-dried or kiln-dried and the moisture content was not to exceed 19 percent.

Roof Survey

Chanute AFB engineers surveyed the roof based on a plan developed by USA-CERL. The purpose of the survey was to determine the location of roof sections at which the maximum deflections existed. Readings were taken at points approximately 20 ft on-center along: (1) the length of the perimeter walls of the wings, (2) the walls of the hallway, and (3) a line that coincided with the center of the span between the perimeter wall and hall walls (Figure 10). The data were then plotted and schematic cross sections drawn to determine the actual deflected shape of the roof under dead load conditions. Table 2 summarizes the data. The maximum deflections were 1.14 in. at Line 8 on the south side of the building's west wing and 1.80 in. at Line 12 on the west side of the north wing. The extent of ponding that can occur is evident when sections are drawn of these critical areas (Figure 11). This information identified critical areas requiring visual inspection.

Visual Inspection

The building was inspected visually to assess the condition of the structural members, connections, and structural environment. A general inspection of the grounds and building exterior was conducted. In some areas, the ground surface around the structure is not sloped to drain water away from the building (Figure 12).

Figures 13 through 16 show the condition of the building elevations and roof overhang. Peeling paint was observed in the panels of wood sheathing on the underside of the roof overhang. This condition is typical around the building perimeter but is most pronounced on the east face. Here, water was observed dripping from the eave (Figure 17) and there is a depression in the ground underneath caused by dripping water (Figure 18). After a heavy rain, water also dripped from three roof locations along the south side of the west wing. These areas are directly over the window of Room 214, at the center window unit in Rooms 209 and 210, and between the first and second windows from the east end of Rooms 207 and 208. At the main entrance to the building on the south face, new plywood had been installed in the overhang and water stains could be observed (Figure 19).

At the three-section window units on the second floor, there appeared to be sag at the top and bottom of the window frame. From the exterior, it was not possible to measure the sag due to the lack of suitable ladders. Sagging at the three bay windows on the first floor was not as noticeable.

The foundation of Building 16 was investigated to verify that the sagging in the area of the three-section window units is not the result of foundation settlement. The pipe tunnel on the south side of the west wing was inspected and all concrete beams at the first floor spanning the tunnel were found to be at constant elevation. No horizontal displacement of the beams from the perimeter wall was observed and therefore it was concluded that foundation settlement does not exist.

Some plugs are missing from holes in the building's exterior siding where the wall cavity has been filled. Figure 20 shows this condition.

The roof survey indicated that several areas are significantly depressed such that ponding can occur and this observation was verified during the visual survey of the roof. The largest deflections in the roof surface are in the area between Lines 7 and 8 on the south side of the west wing and along the west side of the north wing, especially along Line 12, as was determined from the roof survey data. Dark pools of algae had formed in these and other depressed areas. Water could be forced from the roof overhang by stepping on its edge at the point above where the water was observed to be dripping on the ground. Bubbles in the roof surface were noted across the extent of the roof--especially along the line of the perimeter wall on the south side of the building (Figure 21). The roof on both wings had been patched in several locations (Figures 22 through 24) and unpatched tears in the roof surface were also noted (Figure 25).

During another site visit, roof samples were taken in order to determine the number of roof membranes in place. Figures 26 through 31 show the exposed layers of the roof membranes. Two separate roofing systems were discovered. The uppermost system consists of gravel, 5-ply roofing, and approximately 3/4 in. fiberglass insulation. The original roofing system is constructed of gravel and 3-ply roofing. At this time, significant ponding was also observed on the roof (Figures 32 and 33). The metal flashing and gravel stop at the edge of the roof form a barrier about 3/4 in. to 1 in. higher than the roof surface. Consequently, the metal flashing and gravel stop contribute to the ponding problem by allowing an additional layer of water to accumulate on the roof.

Though conditions on the roof might indicate that water had penetrated the building, moisture tests on the roof sheathing, roof joists, floor sheathing, floor joists, wall sheathing, and wall studs at various locations revealed less than 10 percent moisture content. Water stains were observed on the ceiling and walls of the west stairwell and around the roof hatch in this area of the west wing (Figures 34 through 36). The only other visible evidence of water penetration into the building interior was observed over the windows at the ceiling line in Room 214 (Figure 37).

A general survey of the building interior was performed, during which selected areas of the construction were exposed for detailed investigation. Chanute AFB engineers indicated that termites had been a problem in Building 16. Therefore, two openings were made in the first-floor walls near the intersection of the wings to check for termite damage. The first opening was near the basement stairway (Figures 38 and 39) and the second opening was at the entrance to the officers' restroom (Figure 40). The carpenter who opened the holes reported seeing termites and there was evidence of termite damage and trails. Building occupants reported that the south corridor wall adjacent to Rooms 102 through 106 on the first floor of the west wing had experienced extensive termite damage. An opening was cut into the corridor wall of both Rooms 104 and 105 and evidence of past termite activity was present. Since both sides of the walls in the other rooms had been refinished recently, these walls were not opened for observation.

The second floor of the building was originally designed for classrooms; however, all but one of the rooms have been converted to office space by adding partition walls. The remaining classrooms in the west wing, Rooms 216 and 217, contain about 100 desks.

In Room 225A, a crack was observed in the ceiling parallel to the direction of the joists in the center of the room. This crack is almost directly below the point of maximum roof deflection in the north wing. A similar crack exists in Room 225B. Also, the door jambs to Rooms 224 and 225 are distorted, suggesting that the roof sags downward between the two doors. A crack is visible in the ceiling of Room 223. This crack runs from the windows to the center of the room and then diagonally to the southeast corner of the room.

Stained floors were observed in several second-story west wing offices. These stains suggest that file cabinets or other furniture had been in place for a prolonged period. In particular, stains were observed adjacent to the east wall of Room 219 where occupants reported that, in February 1987, the cabinets along this wall had been moved to the west wall (Figure 41). Eight five-drawer file cabinets are now aligned along the west wall and centered about its width (Figure 42).

The floor in Rooms 209 and 210 has major humps and depressions with maximum amplitudes of nearly 5/8 in. (Figure 43). The humps occur directly above the partition walls in the women's restroom on the first floor of the west wing, south side. The maximum deflection occurs between these humps. Room occupants said that these rooms had once been the main recordkeeping area.

Also in Room 210, a stamping machine (Figure 44) sits along the west wall adjacent to the storage closet. This machine weighs several hundred pounds and is used to make identification tags. During normal operation, the stamping machine is quite noisy and induces vibrations into the floor system.

In Rooms 212 and 213, deflections were measured at the top and bottom of the three-section window units in the southeast corner of the room. Deflections of about 0.25 in. were observed at the top and bottom of the window trim and sill (Figure 45).

On the first floor, evidence of heavy second-floor loading was observed in specific areas. Deflection of the ceiling tiles in Room 104 indicates that a significant crack in the plasterboard extends from the south side of the room to the north approximately 1 ft east of the partition wall. Several ceiling tiles had fallen recently, exposing the crack (Figure 46). Room occupants stated that when the stamping machine directly above in Room 210 is in use, the ceilings of Rooms 104 and 105 experience significant vibrations. Overload and vibration due to the stamping machine appear to have caused this crack.

No crack in the ceiling plaster was observed in the hall but another crack extended from the south side of Room 114 to the north. This crack is located under the east wall of Room 219 above. Here, ceiling tiles are also deflecting and separating from the plasterboard (Figure 47). When a few tiles were removed from one area to investigate the structural condition, several tiles fell in other areas along the length of the crack. It appears that the file cabinets that had been located above in Room 219 prior to February had overloaded the floor and caused the crack.

The structural detail for support of the original partition wall on the east end of Room 219 also was observed from Room 114. As stated in the specifications, a joist has been installed under the partition and blocking is provided at 24 in. on-center between the typical floor joist and the added joist. Here, the blocking is not snug between the

typical floor joist and the added joist. Gaps of 1/8 in. on one side and 1/16 in. on the other side are evident at the top of the blocking. At the bottom of the blocking, there are gaps of 3/16 in. and 1/16 in. on the two sides. The blocking has dropped about 3/8 in. below the joists. Figures 48 (detail A) and 49 show this condition.

Support for the original partition wall of Room 211 was investigated from Room 105 below. Figure 48 (detail B) shows the structural detail at this location. In this case, there is a 0.5-in. gap between the top of the added floor joist and the bottom of the deck. This gap appears to run along the entire length of the joist.

Connections at the top of both new and original partition walls were also investigated. In Figure 50, detail A shows this condition for the original walls and detail B shows it for new partition walls.

Upon exposing selected locations in the ceilings of the first and second stories, the structural members were measured. Although nominal joist sizes are 2 in. by 14 in. and 2 in. by 8 in., the actual dimensions should be 1.5 in. by 13.25 in. and 1.5 in. by 7.5 in., respectively. The measured dimensions were 1.5 in. by 13 in. and 1.5 in. by 7.125 in. The joists appear to be slightly undersized, as this variation is more than would be expected due to shrinkage. The wall stud dimensions are as would be expected.

3 STRUCTURAL ANALYSES

Typical roof and floor joists and window header beams were analyzed to assess (1) the type and magnitude of forces applied to the structural elements by the loads and (2) the stresses and deformations produced in the elements by the applied forces.

Allowable Bending Stresses

The design stresses mandated in the carpentry specification were used to establish allowable stresses for the structural analyses. To adjust the specified bending stress for the duration of load, the factors cited in Table 3 were used to establish allowable bending stresses.

Roof Joist Analyses

By inspection, the 2 in. by 8 in. roof joists spanning the hall corridor are not stressed to the same intensity level as the 2 in. by 14 in. roof joists. Therefore, the 2 in. by 14 in. roof joists are critical.

A simple beam spanning 23.5 ft with an overhang of 3.5 ft was used to model the span for the 2 in. by 14 in. roof joists. The tributary width for a typical joist is 1.33 ft. The following roof load combinations were evaluated:

Dead Load (DL)	30.5 plf*
DL + Live Load (LL)	57.2 plf
DL + Snow Load (SL)	49.2 plf
DL + Ponding (W')	1253.1 lb

The ponding load assumes a uniform water load of 1.25 in. and a parabolic load of 1.8 in.

Several assumptions were made in analyzing the roof joists. First, it has been shown that the inelastic deflection, creep, or permanent set of sawed lumber is approximately equal to the calculated dead-load deflection. Therefore, the total long-time deflection may be as much as two times the calculated initial elastic deformation. A factor of 2 was therefore used in these dead-load deflection calculations to reflect the duration of dead load on the roof system. It follows that additional ponding will result from permanent setting of the structural member.

Second, the effect of ponding water on the roof is to magnify deflections and stresses. The ponding magnification factor is expressed by the formula:

$$C_p = \frac{1}{1 - W' L^3/\pi E^4 I} \qquad \text{[Eq 1]}$$

*Pounds per linear foot.

where:

C_p = factor for multiplying stresses and deflections under existing loads to determine stresses and deflections under loads plus ponding and is known as the magnification factor.

W' = total load of 1 in. depth of water on the roof area supported by the joist (lb/in.).

L = span length of joist (in.),

E = modulus of elasticity of joist (psi).

I = moment of inertia of joist ($in.^4$).

For the roof system of Building 16, the magnification factor is 1.34. Values calculated for roof deflections due to dead load and ponding represent measured deflections multiplied by this factor. Stresses due to this load combination are calculated based on the sum of the dead load weight and the weight of water that occurs with the measured deflections multiplied by the ponding factor. A modulus of elasticity of 1600 ksi was used in these calculations. Table 4 summarizes the results of these roof joist analyses.

Floor Joist Analyses

The 2 in. by 14 in. floor joists were analyzed for a simple span of 23.5 ft. The tributary width to a typical joist is 1.0 ft. The following load combinations were analyzed:

DL	15 plf
DL + code* classroom LL (CLL)	55 plf
DL + code office LL (OLL)	65 plf
DL + realistic classroom LL (RCLL)	34 plf
DL + file cabinets (FDL)	115 plf

The weight of a 15 in. by 30 in., five-drawer file cabinet was assumed to be 500 lb. In addition, it was assumed that the floor decking distributed the weight of the cabinets over four floor joists and that eight cabinets were located over the center of the span.

A modulus of elasticity of 1600 ksi was used in analyzing the floor joists. No duration of load factor was considered. Table 5 summarizes the floor joist analyses.

The 2 in. by 8 in. floor joists were analyzed for a simple span of 10.0 ft and a tributary width of 1.0 ft. In addition to dead loads, the joists were analyzed under a load combination of dead load plus a corridor live load of 100 psf. Table 6 summarizes the 2 in. by 8 in. floor joist analyses.

*Refers to Uniform Building Code 198.

Window Header Beam Analyses

The header beams above the second-story windows were analyzed. These beams are constructed of three 2 in. by 10 in. joists. They span 11 ft 3 in. and support roof joists. The load combinations analyzed were: DL; DL + LL; DL + SL; and DL + W'. Table 7 shows the results. Duration of load and ponding magnification factors were used in calculating the stresses and deflections.

Header beams above first-story windows were also analyzed. These beams are constructed of three 2 in. by 12 in. joists. They span 11 ft 3 in. and support floor joists. The load combinations analyzed were: DL; DL + CLL; DL + OLL; DL + RCLL; and DL + FDL. Results from the analyses of the first-story window headers are given in Table 8.

4 DISCUSSION OF FINDINGS

Building 16 was constructed in accordance with the practices and standards prevailing in early 1950, particularly with regard to the use of a flat roof with no slope to facilitate drainage. All structural lumber was dry and sound except for the locations where termite damage was observed. The actual dimensions of the structural lumber were slightly less than those typical for buildings of this age.

Roof

Ponding water on the roof is a major problem with Building 16. The original roof was designed to be a flat system with no slope or pitch. Moreover, no roof drains were provided and a 0.75 to 1 in. high metal flashing and gravel stop were installed around the entire perimeter of the roof, contributing further to the ponding problem. Current design guidance recommends that roof surfaces have a minimum residual slope or camber of not less than 0.25 in./ft of horizontal distance between the level of the drain and any point on the roof to minimize water ponding. This minimum slope should remain after long-term deflection of the members has taken place. (The general problems plaguing flat roofs are described, e.g., in Rosenfield.[1])

When the new roof system was added to Building 16, it increased the dead load on the roof joists and corresponding deflections by about 50 percent. Due to the 3 ft 6 in. overhang, any deflection at mid-span of the roof joist produces an upward deflection at the edge of the overhang. This condition aggravates the ponding problem because it raises the top of the metal flashing at the edge of the overhang higher with respect to the roof joist supports at the building exterior and increases the maximum depth of ponding at the mid-span of the roof joist.

The effect of ponding water on the roof is to increase deflections and stresses. Calculated ponding deflection corresponds with reported observations of about 4 in. Since the weight of water is about 5.2 lb/sq ft/in., the effect of ponding is to create a live load of about 20.8 lb/sq ft and a live load deflection at the mid-span of the roof joist of about 0.75 in. This deflection, plus that of the dead load, are believed to be responsible in part for the cracks observed in the second-story ceilings. These deflections will recur any time there is enough rain to create extensive ponding on the roof.

Based on this analysis, the reasons for the severe ponding problems on Building 16 are lack of slope, inadequate drainage, and additional dead loads.

Second-Story Floor Deflections

In the original design, the second story of Building 16 was used for classrooms almost exclusively. The design live load for classrooms was probably 40 lb/sq ft. The second story is now almost totally allocated to office space. The current design live load for office space is 50 lb/sq ft. However, there are numerous stains on the second floor, suggesting that files or other cabinets have been in place for a prolonged period. For

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[1] M. J. Rosenfield and C. Doyle, *Sloped Roof Conversions for Small Flat-Roof Buildings*, Technical Report M-85/05/ADA154519 (U.S. Army Construction Engineering Research Laboratory, December 1984).

example, in Room 219, there are eight five-drawer file cabinets along the west wall of the room. Until February 1987, these file cabinets had been located along the east wall of the room, next to the storage closet. Directly below the east wall of Room 219, in Rooms 114 and 115, there is a crack across the ceiling parallel to the direction of the floor joists. It is believed that overloading due to the weight of the file cabinets along the east wall of Room 219 is in part responsible for the ceiling crack in Rooms 114 and 115. A similar problem does not appear with the file cabinets along the west wall of Room 219 because there is a 2 in. by 4 in. load bearing partition wall directly under this wall which provides considerable support for this area.

In Rooms 209 and 210, there are two major humps in the floor that coincide with the load bearing walls of the women's restroom on the south side of the first floor. The maximum deflection between the two humps is about 5/8 in. The exact cause for such a large deflection is unknown. Perhaps this area of the room was heavily overloaded with file cabinets or file drawers in the past. If so, the floor joists would transfer a major portion of the load to the header beam over the 11 ft wide first-floor window. If the overload had remained in place for an extended period, creep may have occurred in both the header beam and the floor joists which would have resulted in permanent deflection when the load was removed.

Window Header Beams

The original drawings indicate that the header beam above the second-story three-section window units consist of three 2 in. by 10 in. members with a span of about 11 ft 3 in. The header beams carry the roof loads. The structural analysis of the header beam revealed that the current dead loads will produce a deflection of nearly 0.30 in. which corresponds reasonably well with the 0.25-in. deflection observed at the top and bottom of the southeast window in Rooms 212 and 213. Moreover, when ponding was considered in the structural analysis, the combined dead and live load stresses in the header beam exceeded the allowable stress by about 52 percent.

The added dead load from the second roof system, ponding, and creep are thought to be the reasons for the sagging observed at the second-story three-section window units.

Above the first-story three-section window units, the drawings indicated that the header beam consists of three 2 in. by 12 in. members with a span of about 11 ft 3 in. These header beams support the second-story joists which are spaced at 12 in. on-center. The structural analyses of this header beam revealed that the deflections due to dead load would be less than 0.14 in. Even with an office live load of 50 lb/sq ft, the deflections would be about 0.40 in. and the combined dead and live load stresses would exceed the allowable stress only by about 25 percent. Consequently, sagging above and below the first-floor windows is not as evident as it is for the second-floor three-section window units.

Building Replacement Cost

The replacement cost for Building 16 was estimated based on Army Regulation 415-17, *Cost Estimating for Military Programming* (15 March 1980) and the Building Cost Index published in the 9 July 87 issue of *Engineering News Record*. If Building 16 is assumed to be an administration building three stories or less in height, the current replacement cost (July 1987) would be about $56.20/sq ft for a total replacement cost of

$2.25M. If Building 16 is assumed to be a Post Headquarters building, the current replacement cost would be about $75.25/sq ft or a total of about $3.01M for a 40,050-sq ft building. These estimates do not include an allowance for demolition of the existing building.

New Pitched Roof Cost

The cost of installing a new pitched roof on the remainder of Building 16 was estimated. The roof would have a pitch of 4 in. on 12 in. and be made of wooden trusses, plywood, and conventional shingles. The estimated construction cost for this roof is about $200K. Since the estimated cost of a new pitched roof is less than 10 percent of the replacement cost estimated for the building, most codes would not require that the building be upgraded to comply with current criteria.

Options for Building 16

There are four options for the disposition of Building 16, each of which is somewhat dependent and constrained by available space and funding. The options are:

1. Vacate Building 16 immediately.

2. Construct a new pitched roof and continue to occupy Building 16.

3. Install roof drains and phase out Building 16.

4. Take no action.

Since there appears to be no immediate hazard to the occupants of Building 16, except for that of occasional falling acoustical tile, Option 1 does not seem warranted. Option 2 would require construction of a new pitched roof over the original roof and removal of the current roof system to reduce the dead load on the second-story ceiling. During construction of the new pitched roof, the occupants of the second floor of Building 16, as a minimum, would have to vacate the building. This would cause disruption of normal operations for some period of time. However, with a new pitched roof it might be possible to extend the life of Building 16 another 10 to 15 years. Option 3 would require the installation of roof drains and phasing out of Building 16 by 1992 or sooner. The roof drains would relieve the ponding problem and periodic inspection for additional signs of distress or termite damage should ensure adequate safety. Under Option 4, no action would be taken, which is unacceptable. The ponding problem and related overloads will only continue to make conditions worse and there is a potential for structural failure.

5 CONCLUSIONS AND RECOMMENDATIONS

USA-CERL has evaluated the structural condition of the remaining portion of Building 16 at Chanute AFB, IL. Building 16 appears to have sustained no structural damage that presents an immediate hazard to building occupants or renders the building unsafe except for that of an occasional falling acoustical tile. (Termite damage was not investigated and is unknown.) However, the building is 36 years old and is known to have a termite problem. In addition, the primary use has been changed from classroom to office space, ponding water on the roof is a major problem, and the new roof system has greatly increased the roof dead loads and aggravated the ponding problem. Thus, there is a potential for structural failure, particularly if the ponding water is not removed from the roof or if files and storage cabinets are rearranged in such a way that they overload the second-floor joists and window header beams.

Based on this analysis, the following actions are recommended:

1. Building 16 should be phased out as soon as a suitable replacement structure can be located, rented, or constructed. Although there is no immediate hazard to the occupants of Building 16, the potential for a failure exists. Conditions will only worsen if no action is taken.

2. Roof drains should be installed at numerous locations on the existing roof to control the ponding problem. Until such time that the roof drains are installed, the ponded water should be pumped from the roof following heavy rains that would create extensive ponding.

3. The stamping machine for making identification tags should be moved to the first floor.

4. Until Building 16 is vacated, it should be inspected quarterly to assess any additional evidence of distress and termite activity and to ensure that the roof drains are not clogged with leaves or other debris.

METRIC CONVERSION CHART

1 in. = 25.4 mm
1 ft = 0.305 m
1 sq ft = 0.093 m^2

CITED REFERENCES

Army Regulation 415-17, *Cost Estimating for Military Programming* (Headquarters, U.S. Department of the Army, 15 March 1980).

Engineering News Record (9 July 1987).

Rosenfield, M. J., and C. Doyle, *Sloped Roof Conversions for Small Flat-Roof Buildings*, Technical Report M-85/05/ADA154519 (U.S. Army Construction Engineering Research Laboratory, December 1984).

UNCITED REFERENCES

Evaluation, Maintenance and Upgrading of Wood Structures (American Society of Civil Engineers, 1982).

Gurfinkel, G., *Wood Engineering* (University of Illinois Press, 1981).

Kuenzi, E. W., and B. Bohannan, "Increases in Deflection and Stresses Caused by Ponding of Water on Roofs," *Forest Products Journal* (September 1964).

Minimum Design Loads for Buildings and Other Structures (American National Standards Institute, Inc., 10 March 1982).

Timber Construction Manual, 3rd ed. (American Institute of Timber Construction, 1985).

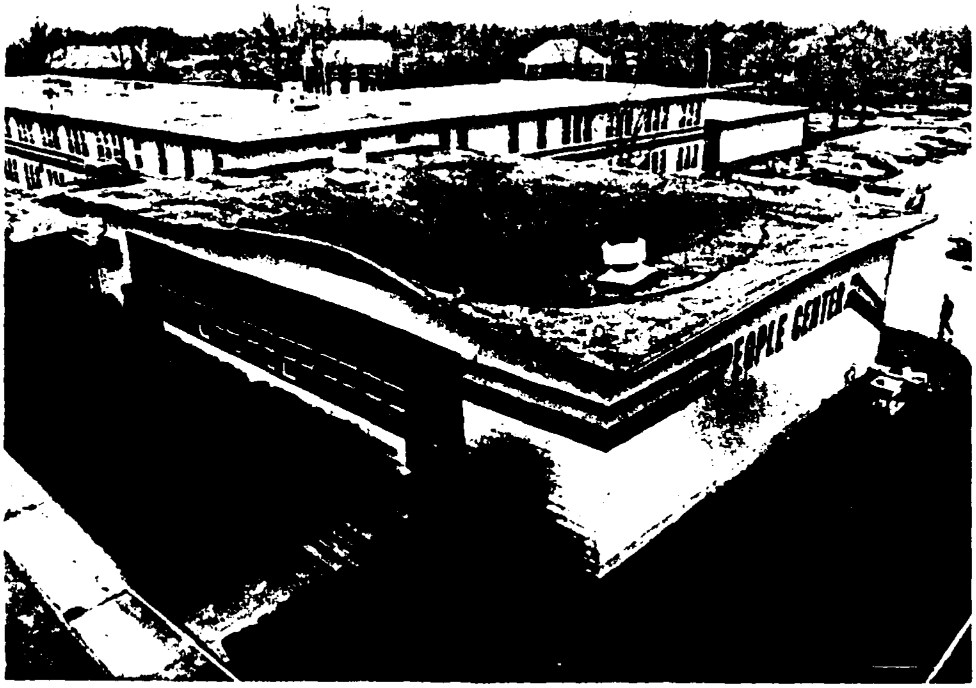

Figure 1. Ponding on the roof of Building 16.

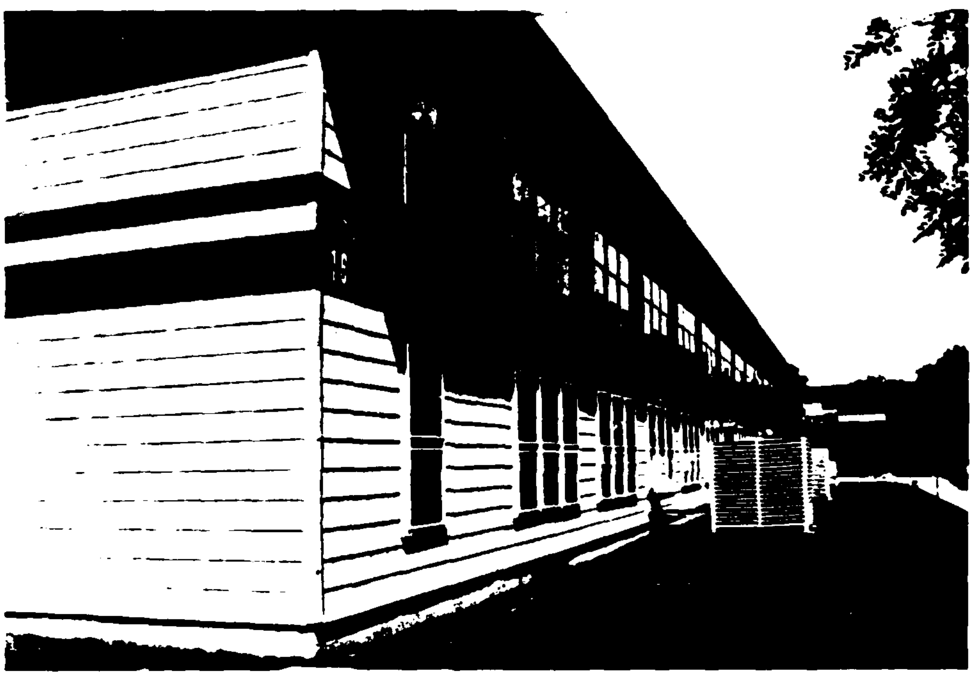

Figure 2. Building 16, west wing, south elevation.

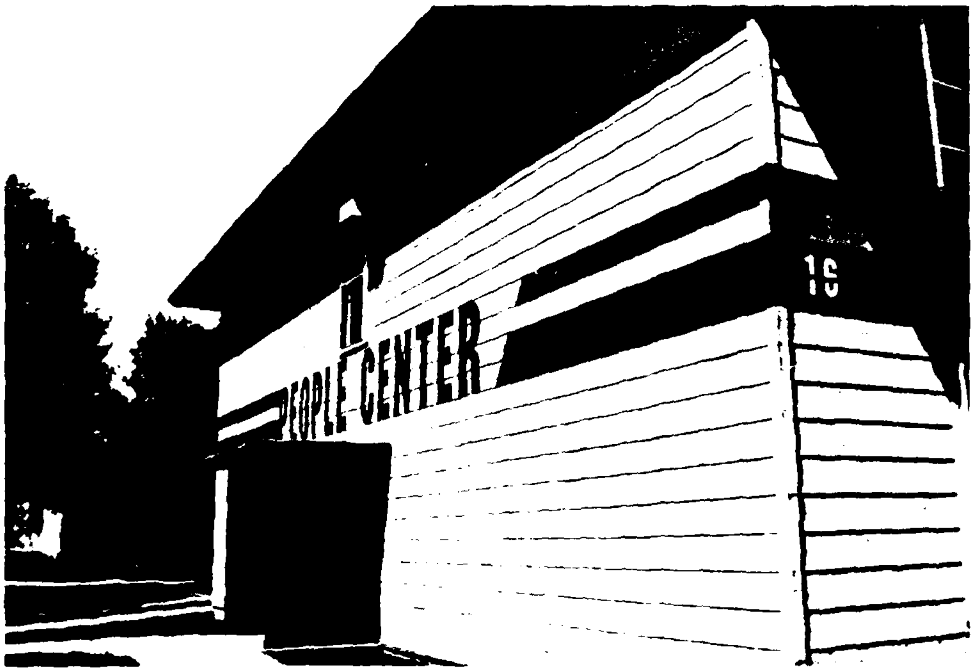

Figure 3. Building 16, west wing, west elevation.

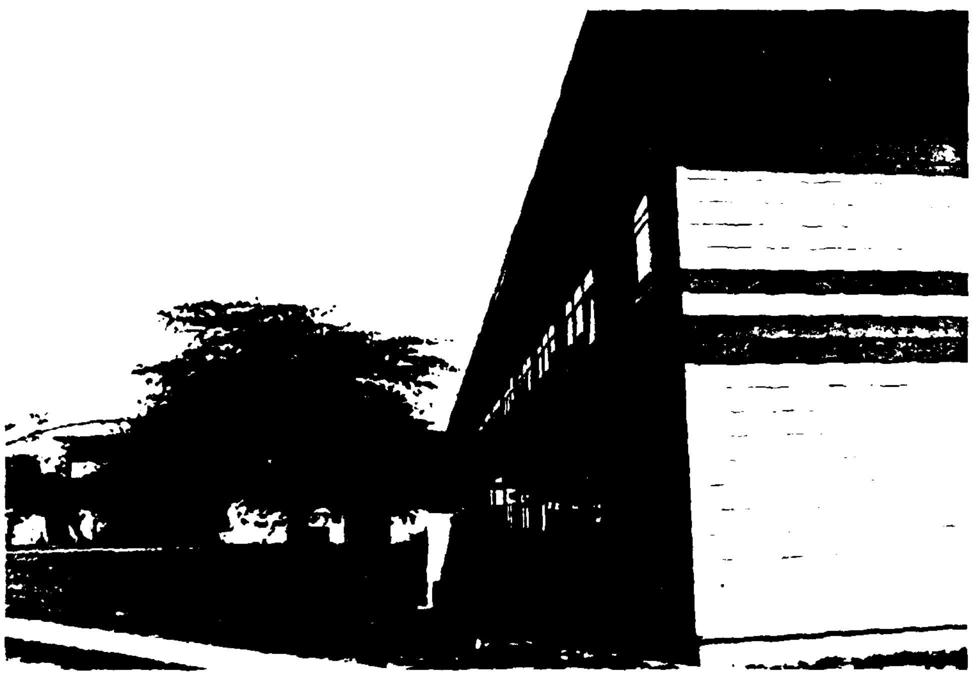

Figure 4. Building 16, west wing, north elevation.

Figure 5. Building 16, north wing, east elevation.

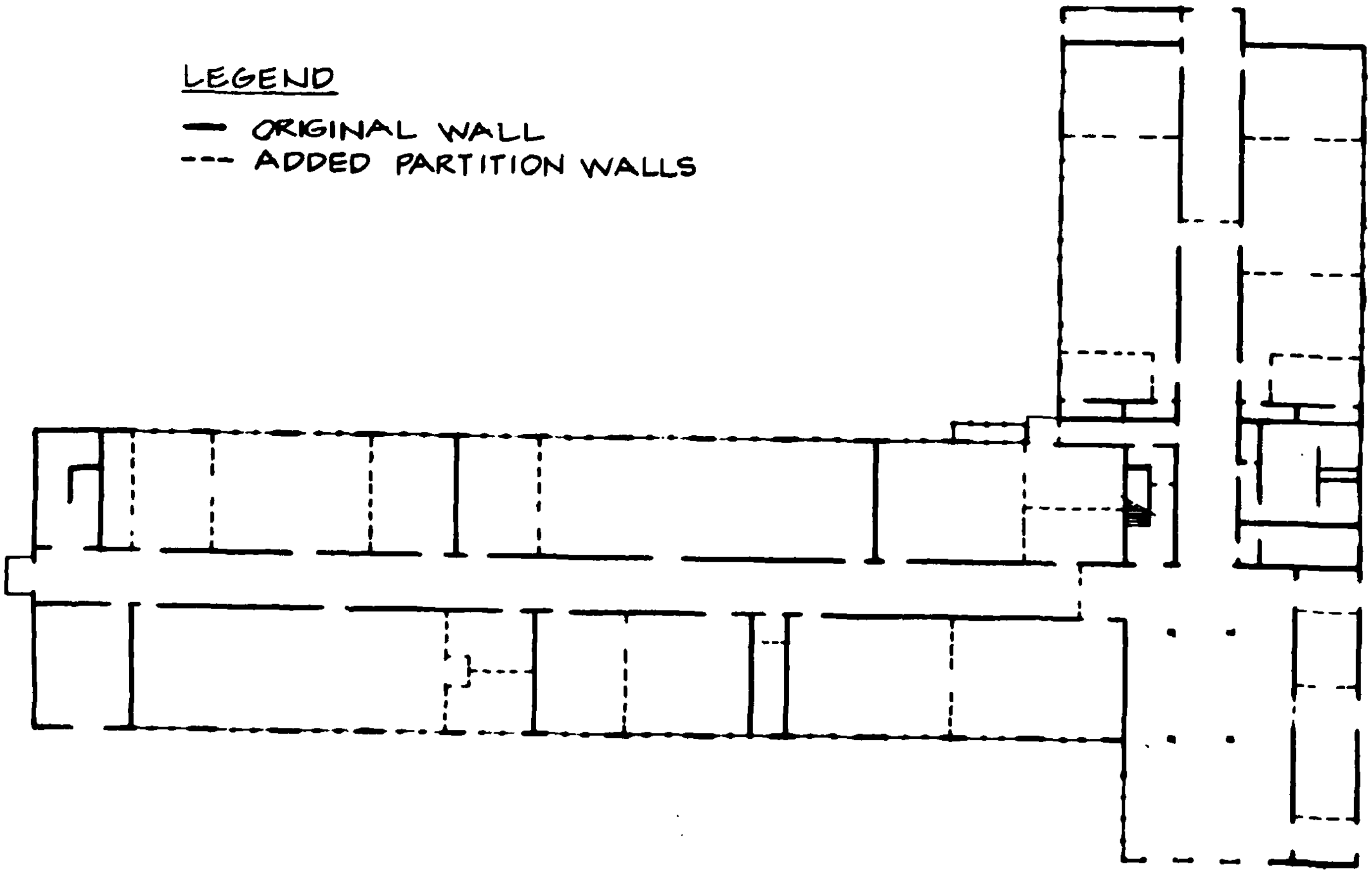

Figure 6. First-floor plan.

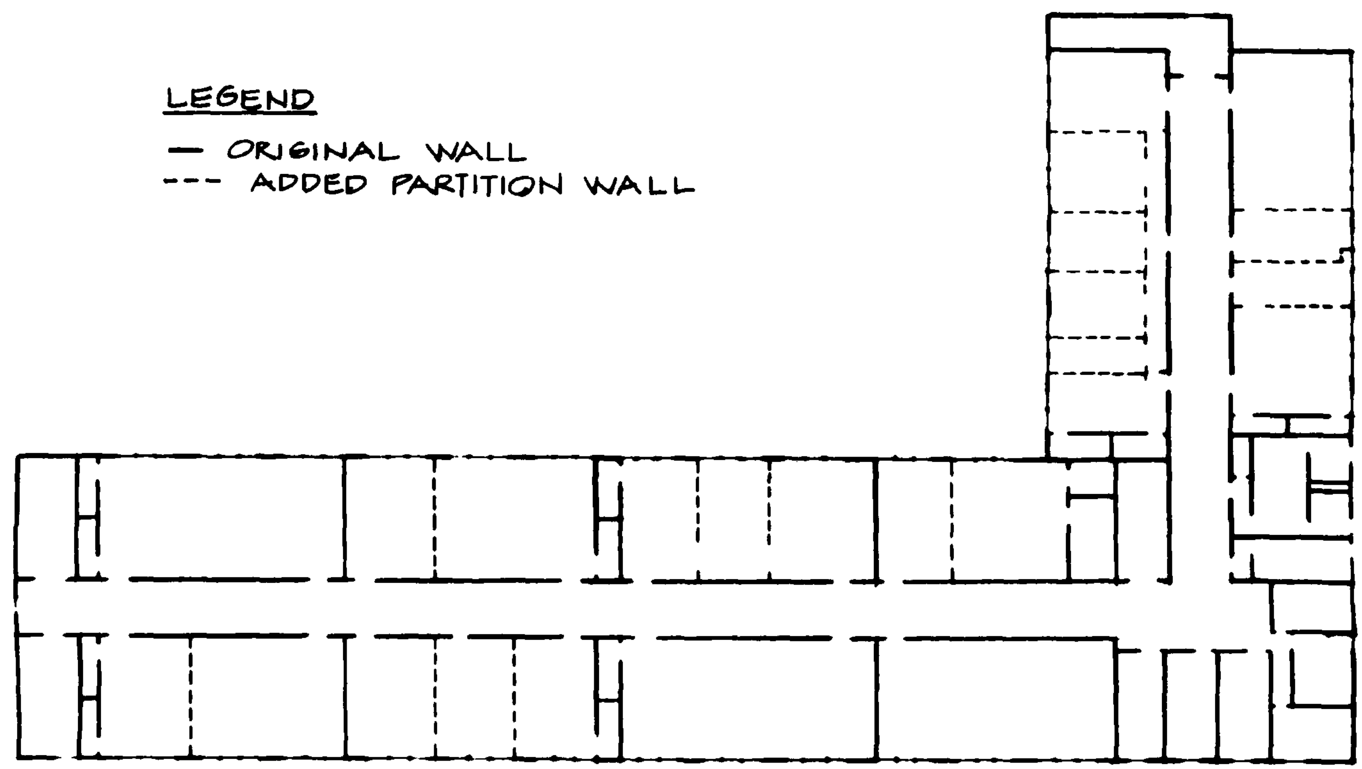

Figure 7. Second-floor plan.

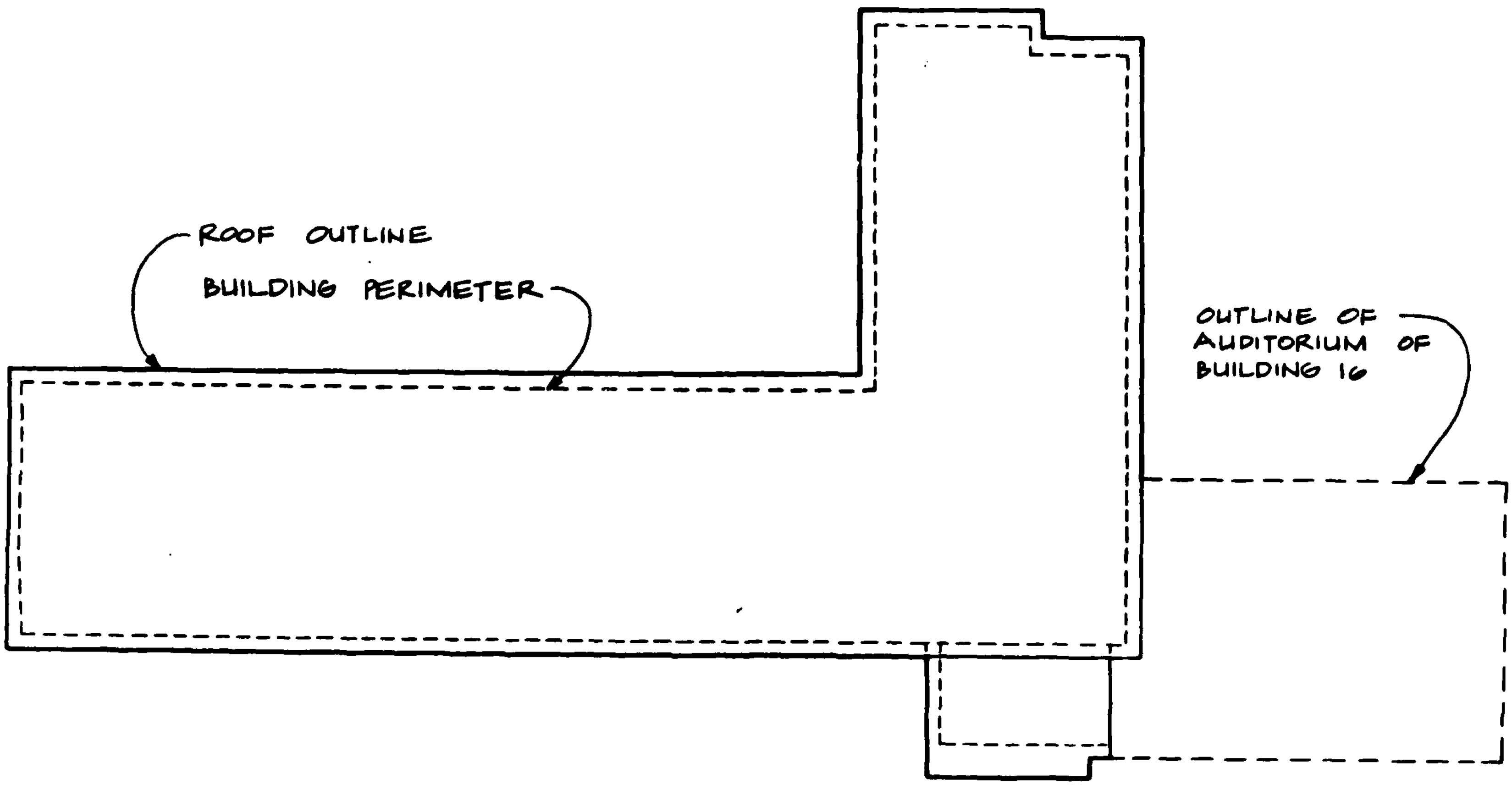

Figure 8. Roof plan.

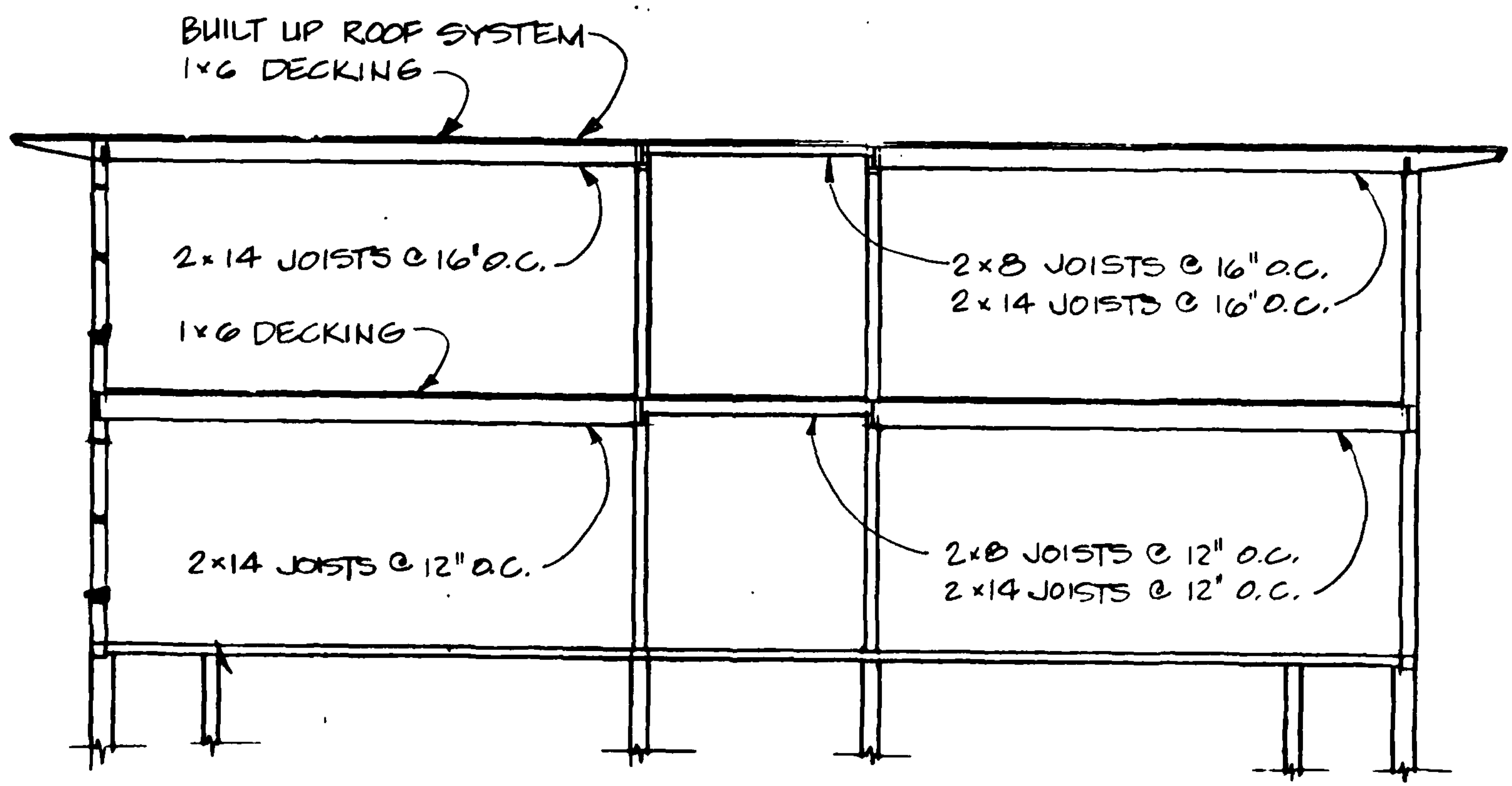

Figure 9. Typical cross section.

Table 1

Original Carpentry Specifications

Component	Specification
Trusses, columns, joists, stair stringers, stair carriages, and similar structural members	Grade: Common Structural Larch or Douglas Fir, No. 1 Southern Pine F_b: 1450 psi E_c: 1600 ksi E : 1600 ksi
Studs, plates and nominal 2-in. roof decking	Grade: No. 1 dimension
Subflooring, wall sheathing, and roof sheathing (not including roof decking)	Grade: No. 1 Boards-Douglas Fir coast region, No. 2 Boards-Southern Pine, No. 3 Boards-Douglas Fir inland region

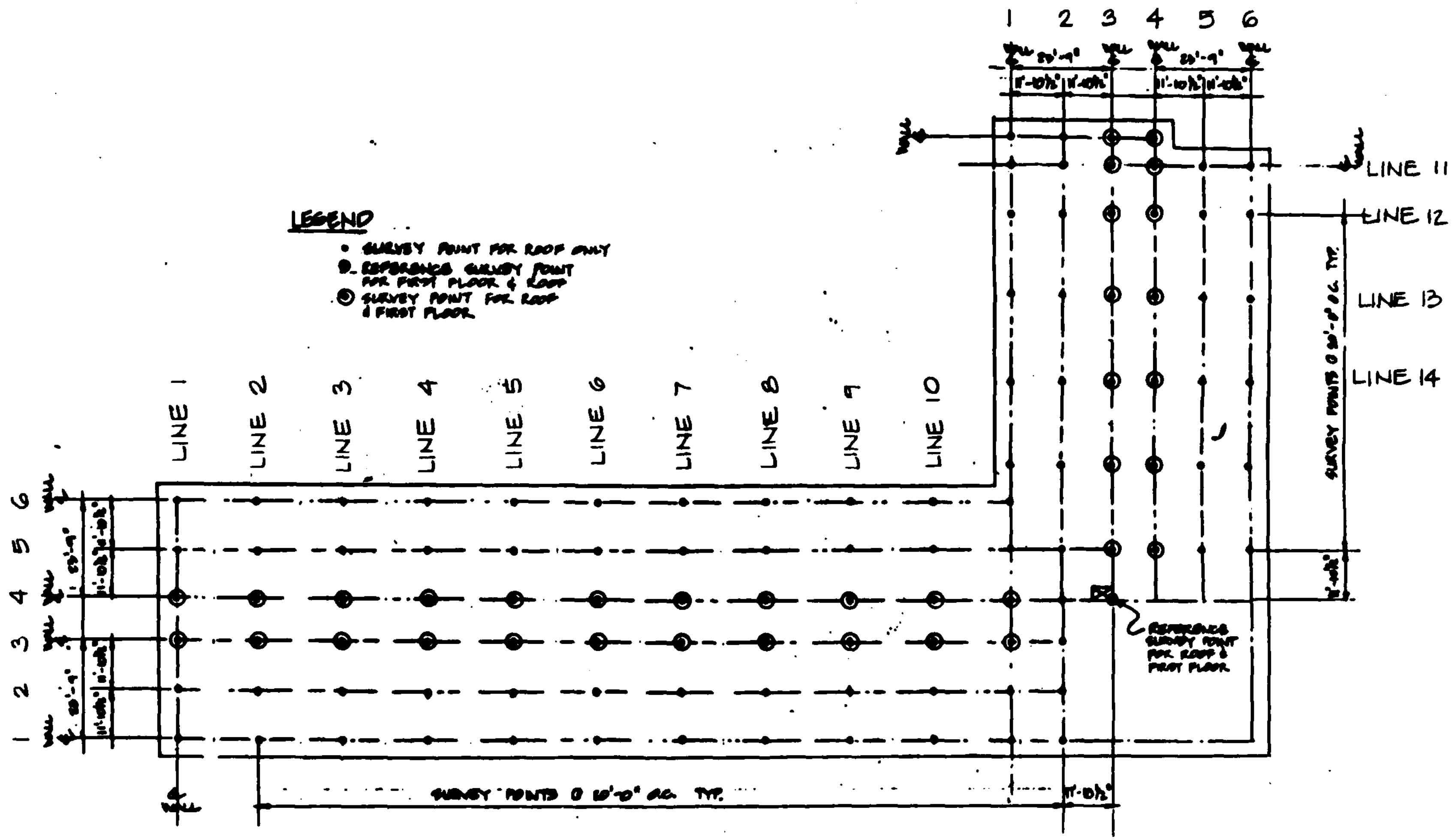

Figure 10. Roof survey plan.

Table 2

Roof Survey Data

Line No.	Column Line						Midspan Deflection (ft)	
	1	2	3	4	5	6	brv 1-3	brv 4-6
1	5.34	5.30	5.29	5.26	5.26	5.24	0.015	-0.010
2	5.40	5.41	5.38	5.35	5.38	5.28	-0.020	-0.065
3	5.40	5.42	5.36	5.36	5.41	5.33	-0.040	-0.065
4	5.37	5.45	5.36	5.35	5.40	5.36	-0.085	-0.045
5	5.36	5.42	5.36	5.34	5.37	5.32	-0.060	-0.040
6	5.36	5.40	5.38	5.36	5.42	5.35	-0.030	-0.065
7	5.38	5.47	5.38	5.37	5.41	5.37	-0.090	-0.040
8	5.37	5.45	5.34	5.33	5.39	5.32	-0.095	-0.065
9	5.38	5.44	5.39	5.34	5.38	5.32	-0.055	-0.050
10	5.39	5.45	5.36	5.38	5.44	5.35	-0.075	-0.075
11	5.28	5.29	5.28	5.24	5.31	5.32	-0.010	-0.030
12	5.32	5.46	5.30	5.34	5.42	5.46	-0.150	-0.020
13	5.34	5.47	5.37	5.33	5.39	5.45	-0.115	0.00
14	5.36	5.48	5.41	5.40	5.46	5.33	-0.095	-0.095

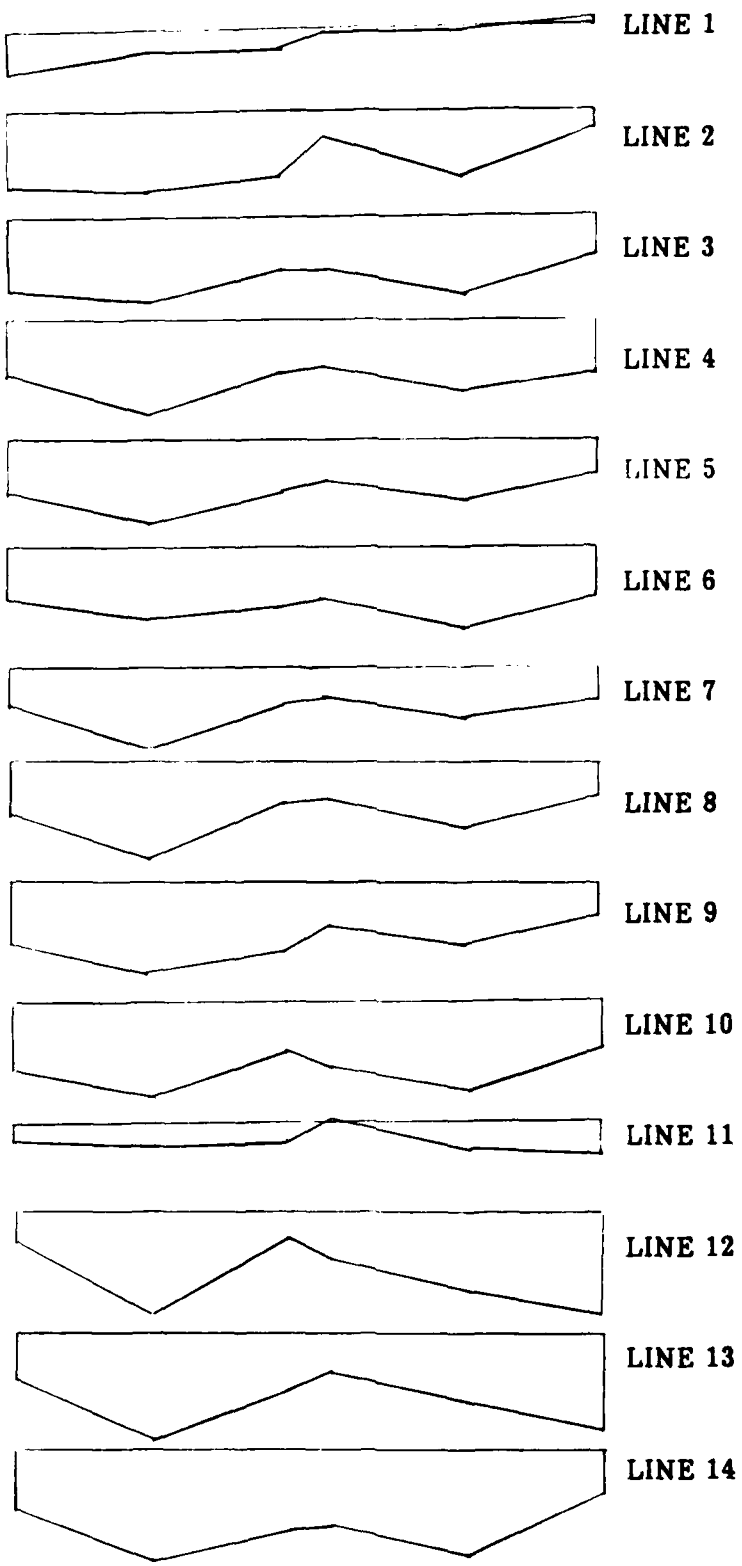

Figure 11. Roof sections based on roof survey data.

Figure 12. Ground slope around Building 16.

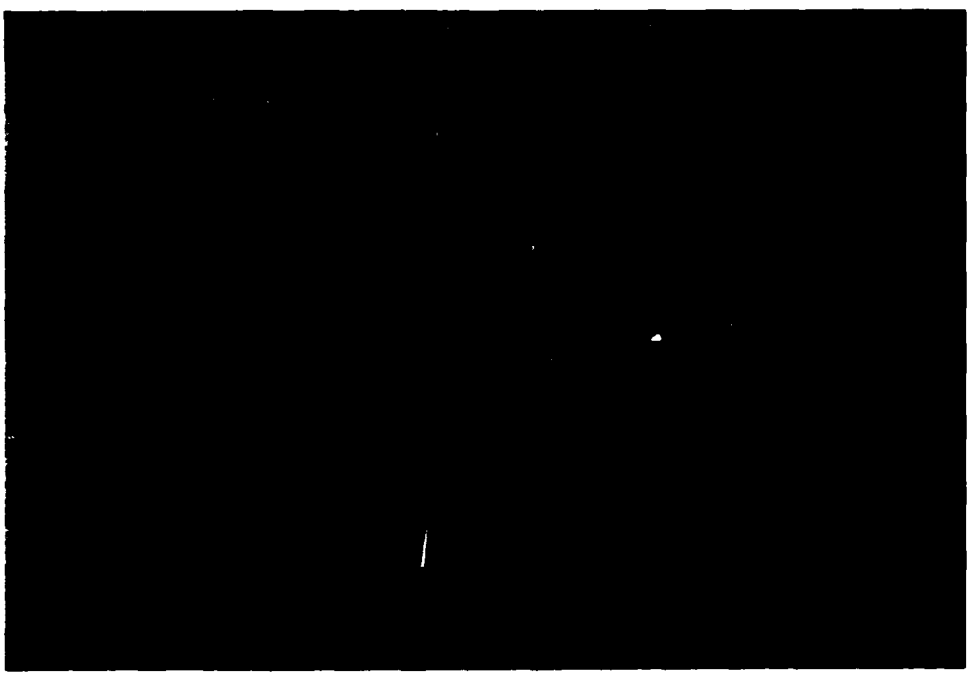

Figure 13. Detail of exterior wall.

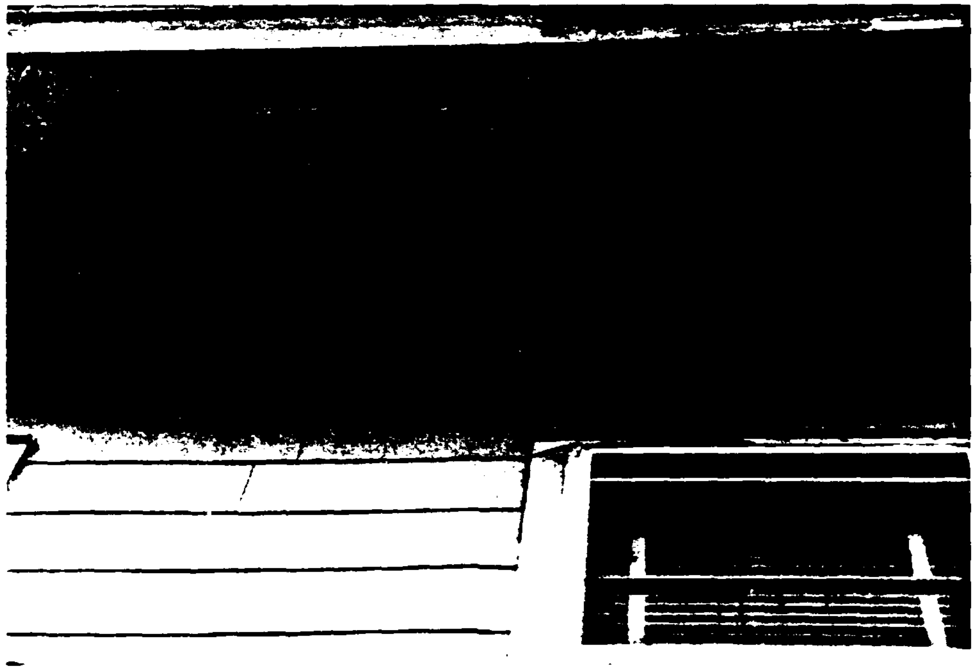

Figure 14. Underside of roof overhang-view 1.

Figure 15. Underside of roof overhang-view 2.

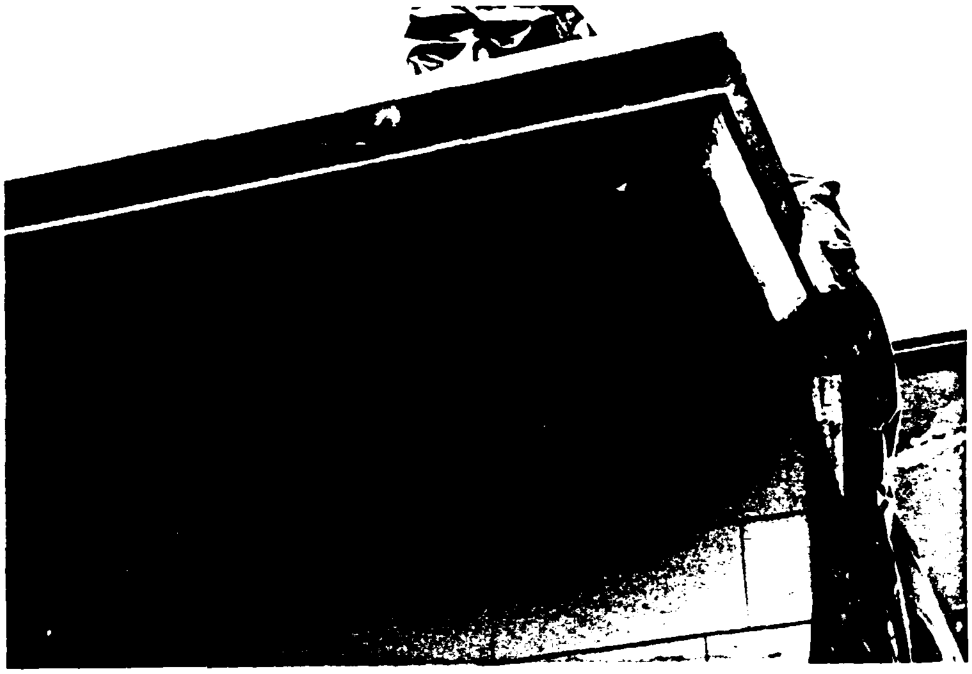

Figure 16. Underside of roof overhang corner.

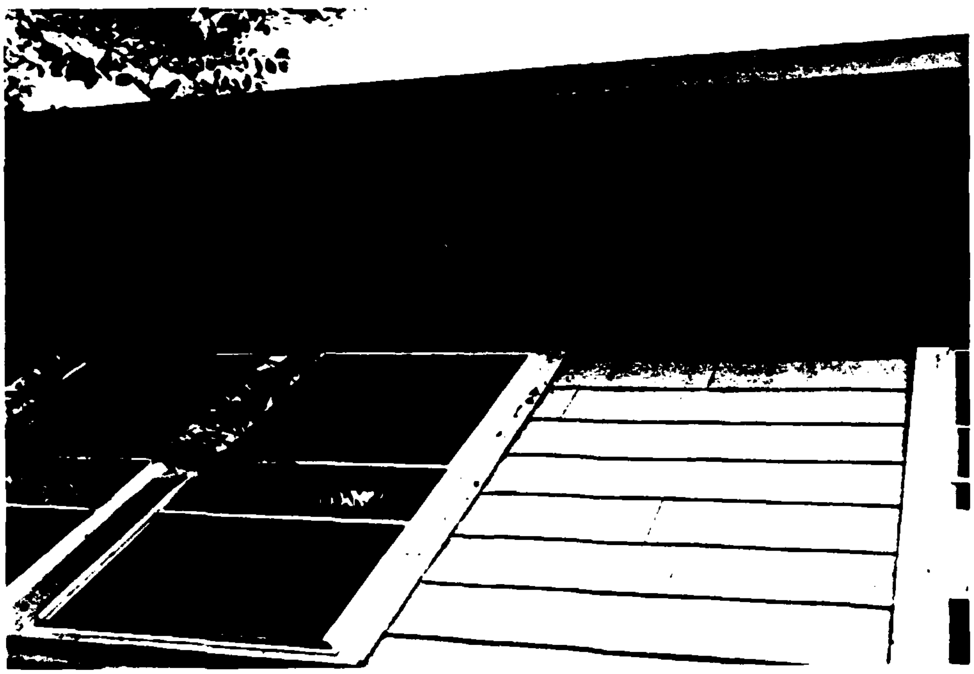

Figure 17. Underside of roof overhang on east side of north wing.

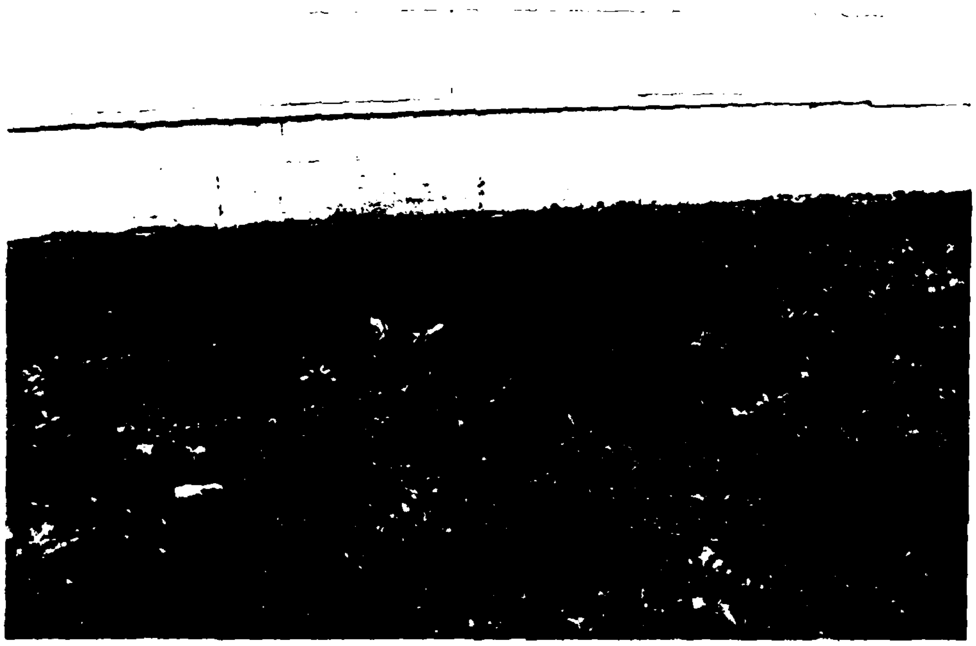

Figure 18. Wet ground at point below roof of Figure 17.

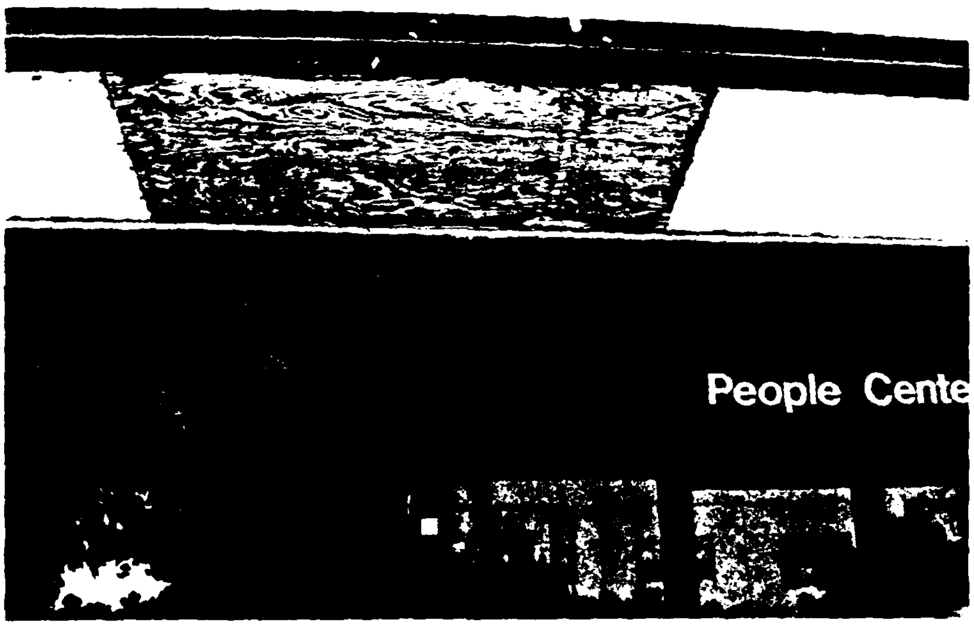

Figure 19. Stained plywood at entrance to Building 16.

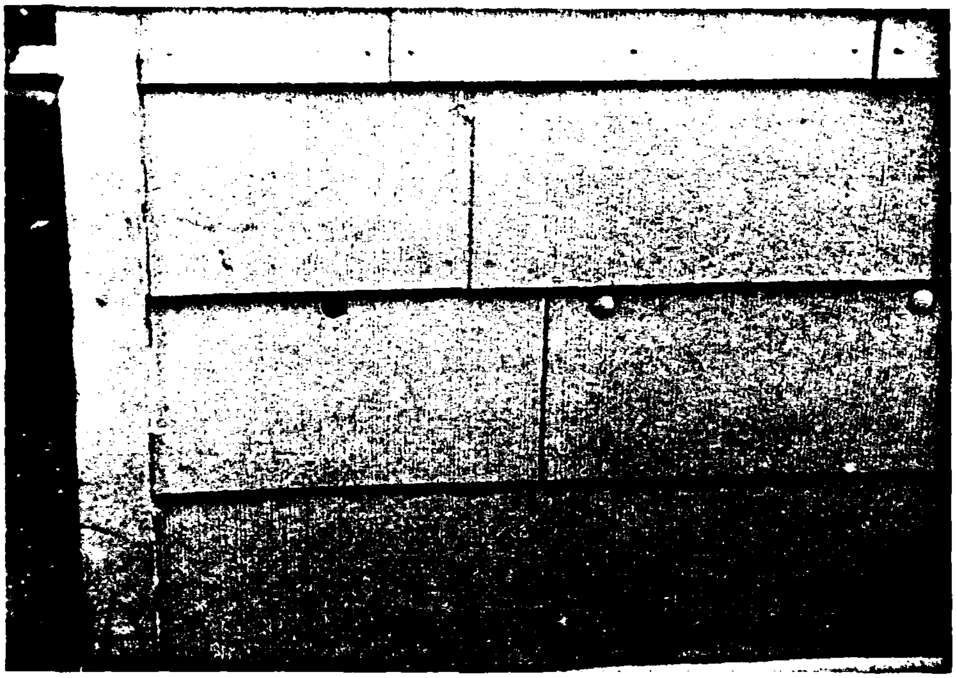

Figure 20. Missing plug on exterior siding.

Figure 21. Bubbles in roof surface.

Figure 22. Patch in roof surface.

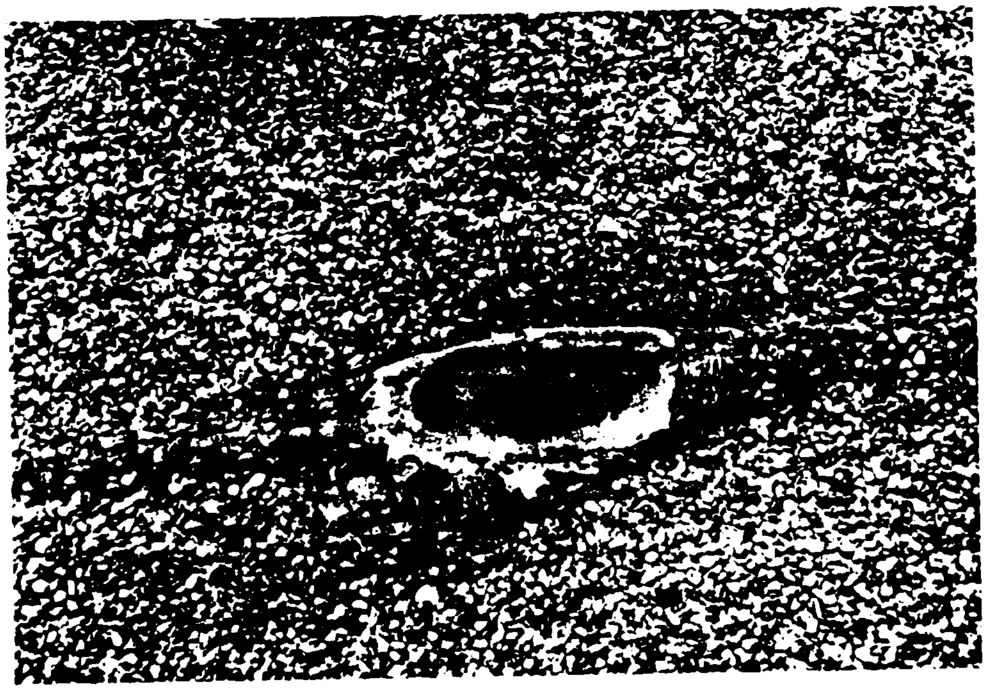

Figure 23. Detail of patch in roof surface.

Figure 24. Several patches in roof surface.

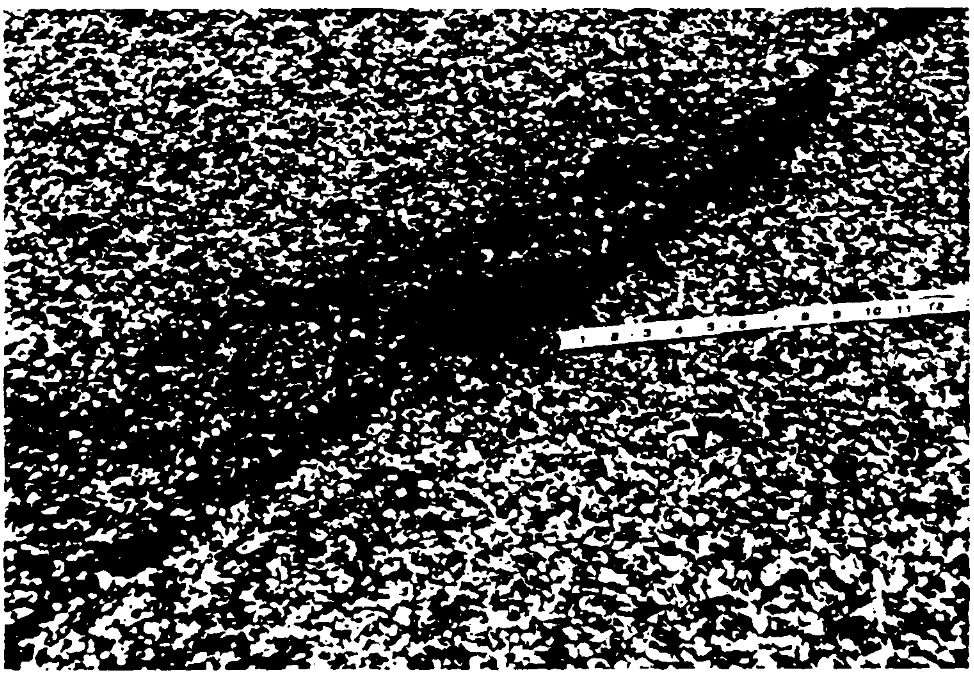

Figure 25. Unpatched tears in roof surface.

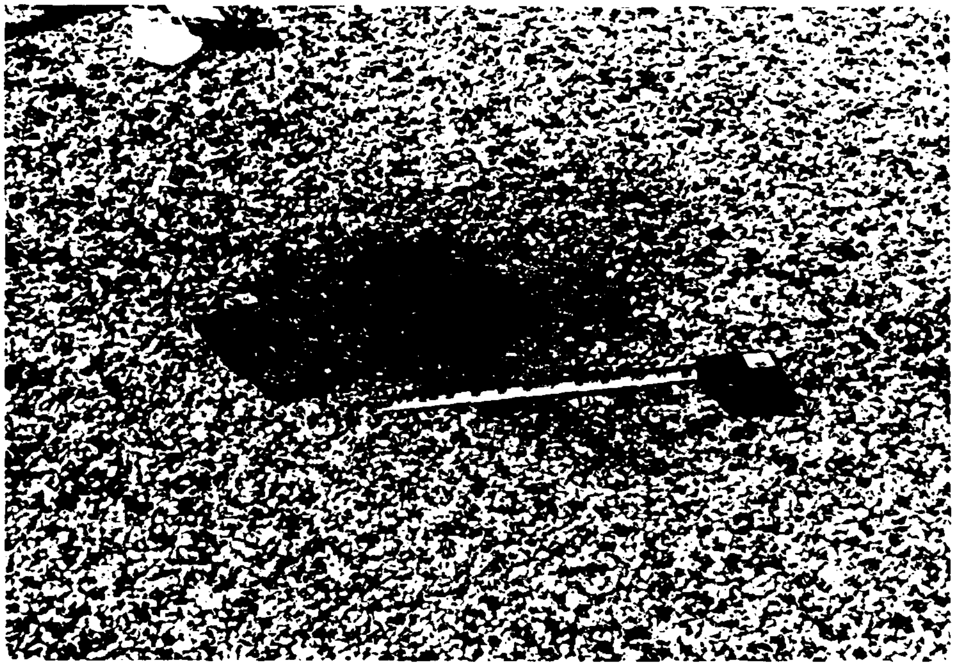

Figure 26. Removal of uppermost roofing system.

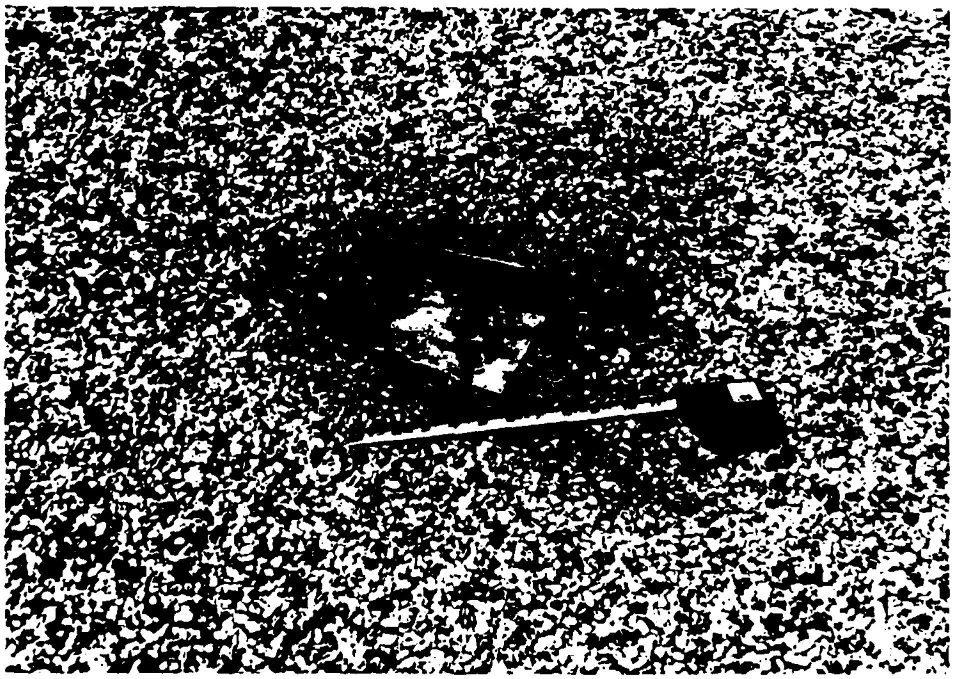

Figure 27. Exposed layer of fiberglass insulation in roofing system.

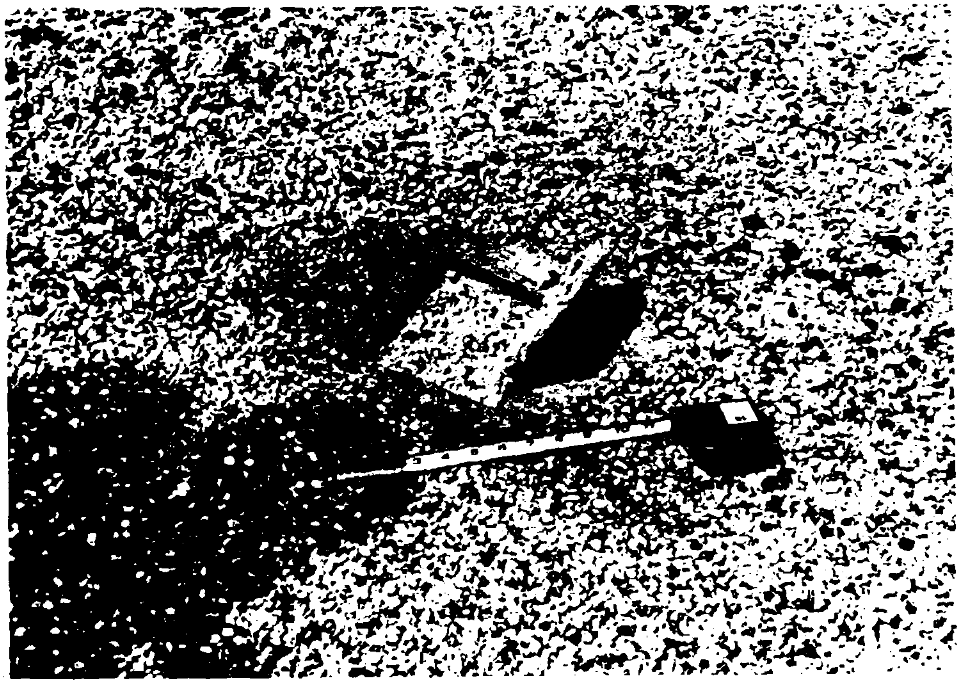

Figure 28. Exposed layer of gravel of original roofing system.

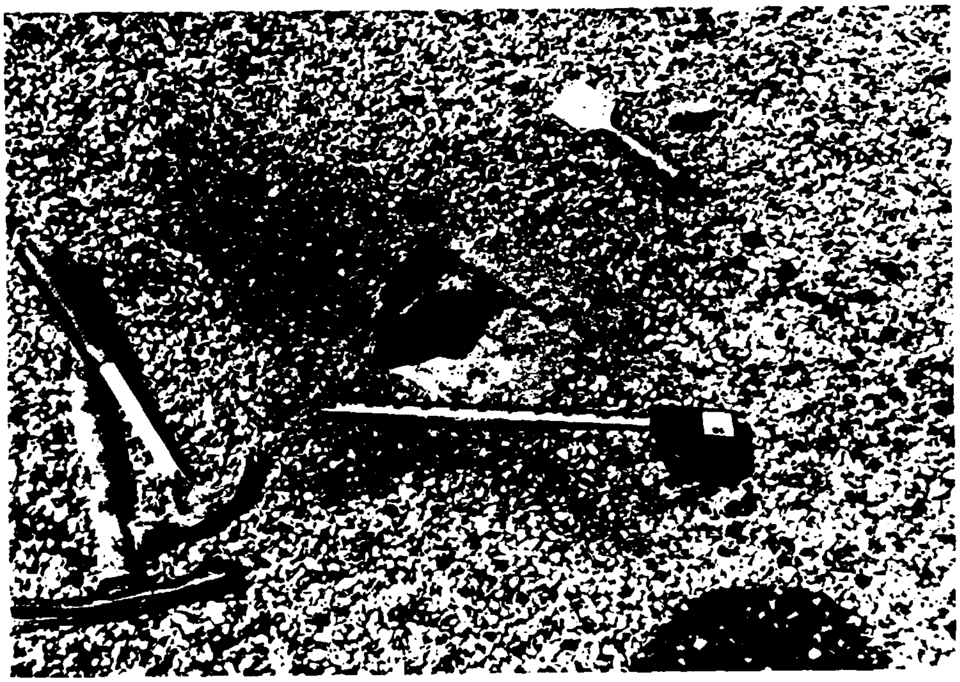

Figure 29. Exposed layer of original roofing system.

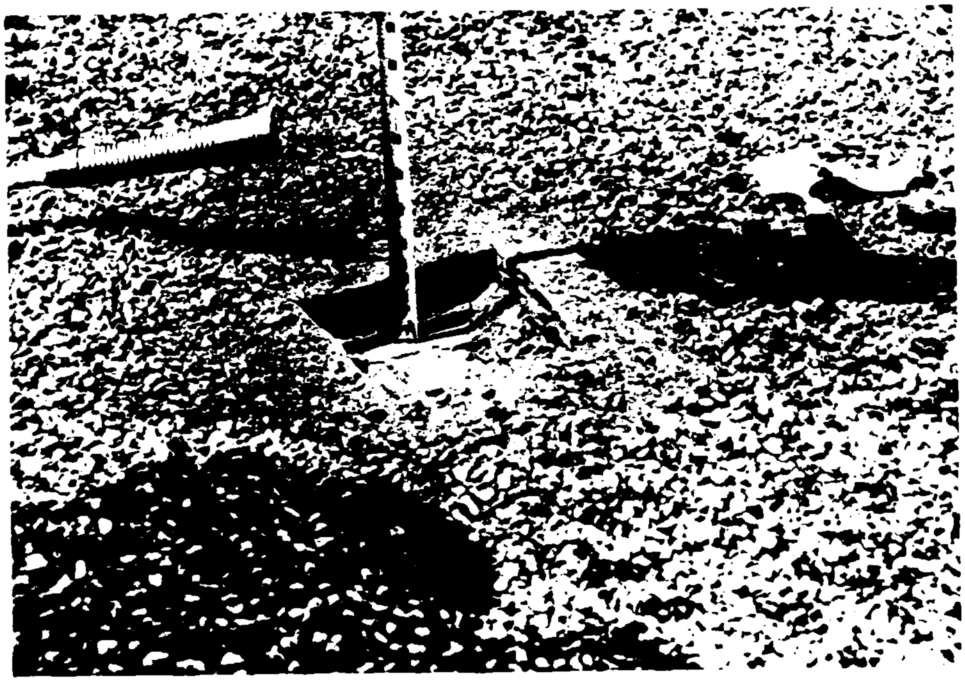

Figure 30. Exposed roof deck.

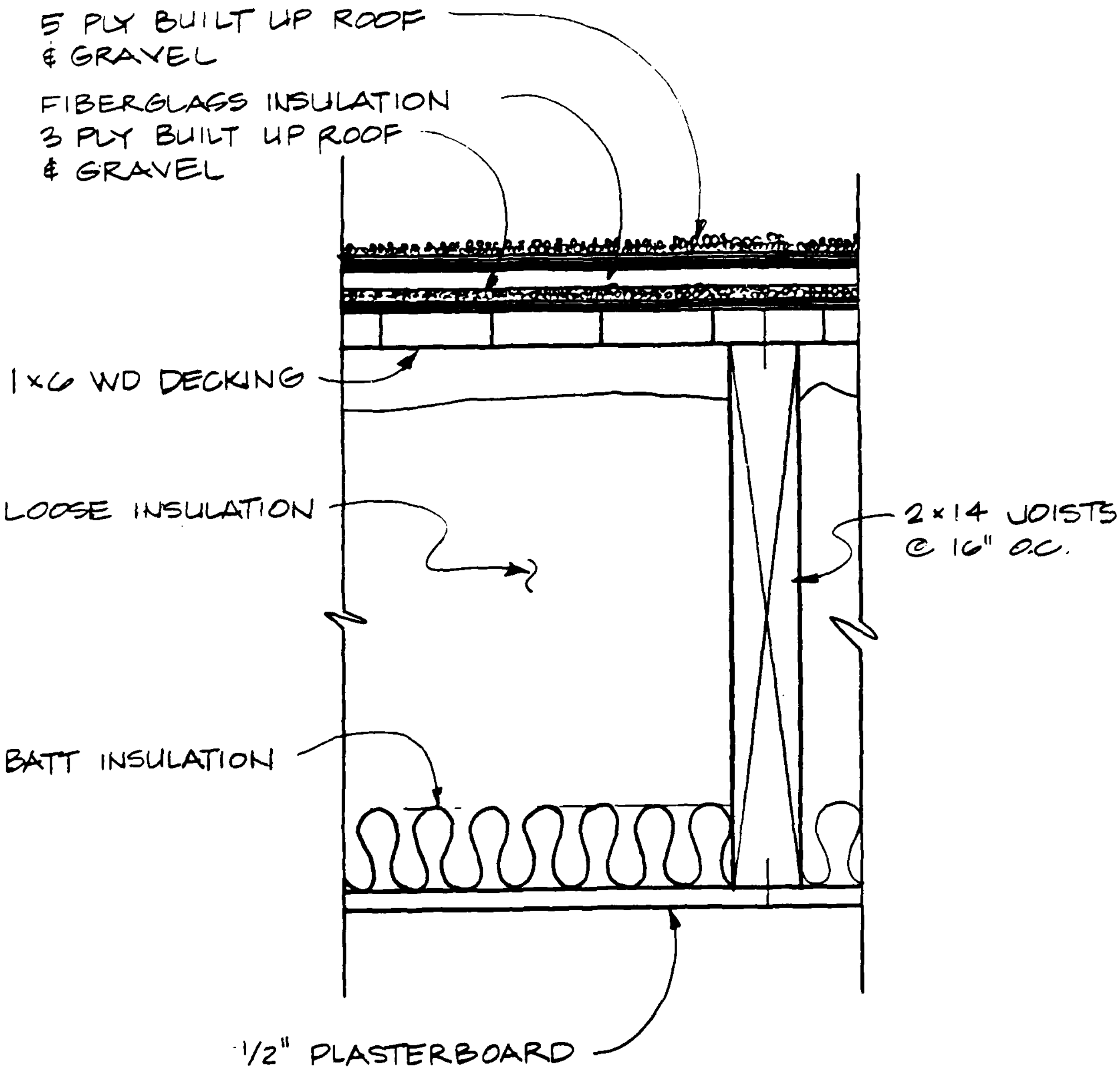

Figure 31. Roof system detail.

Figure 32. Ponded water on south side of west wing.

Figure 33. Ponded water on west side of north wing.

Figure 34. Water staining on wall in west stairwell of west wing.

Figure 35. Water staining on ceiling in west stairwell of west wing.

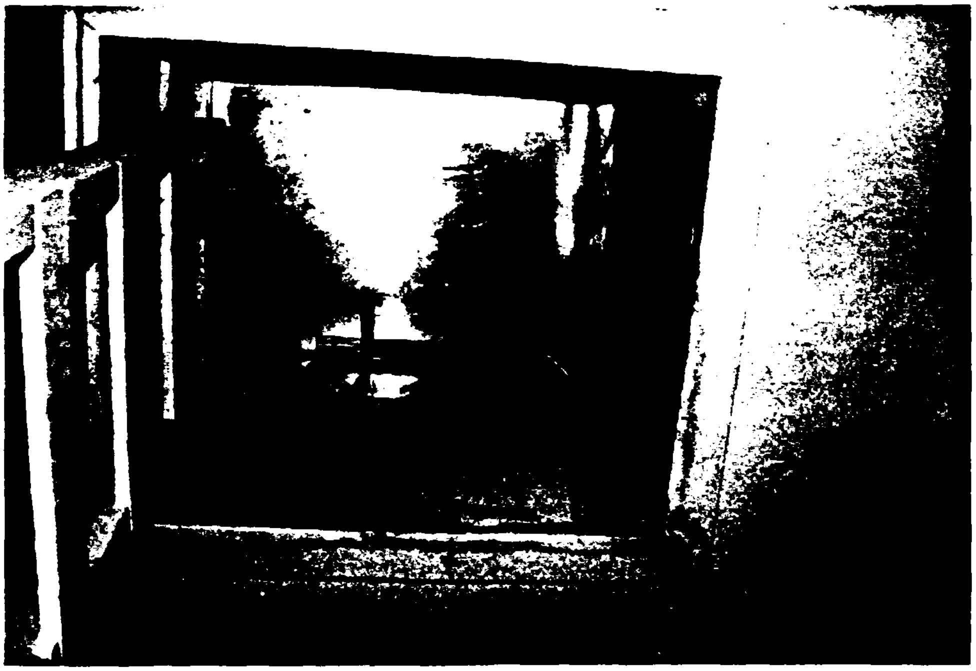

Figure 36. Roof hatch in west stairwell of west wing.

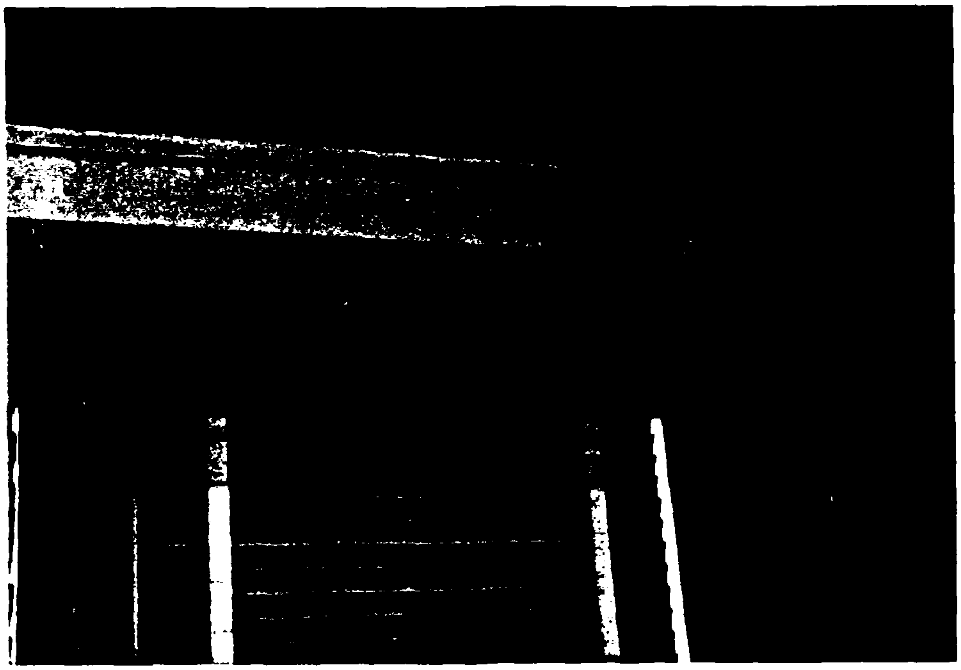

Figure 37. Water staining on ceiling of Room 214.

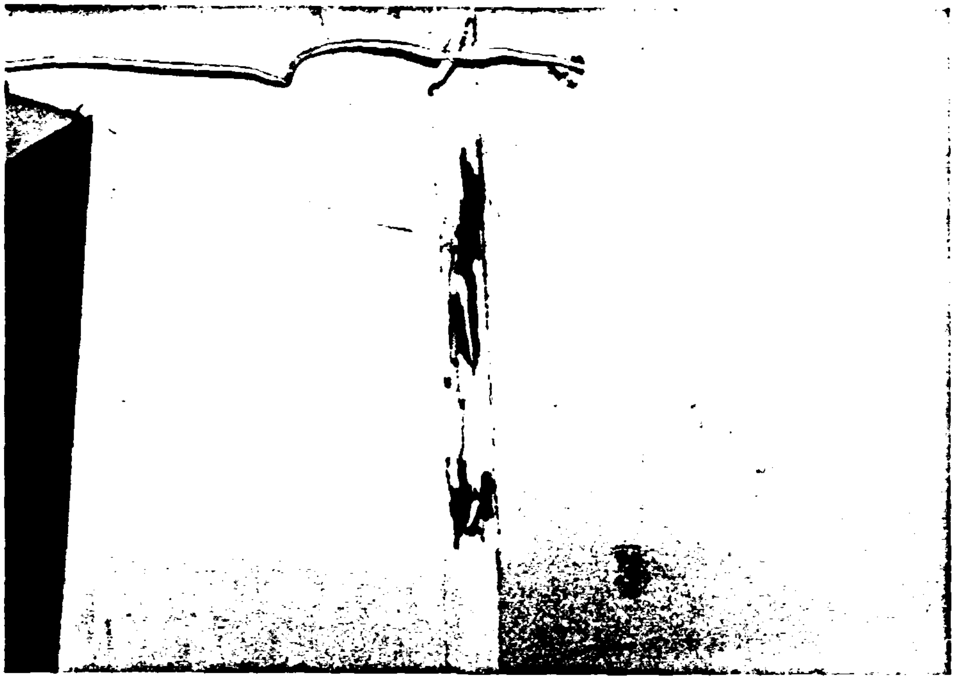

Figure 38. Termite damage near vending machine hall.

Figure 39. Detail of termite damage near vending machine hall.

Figure 40. Termite damage near officer's restroom.

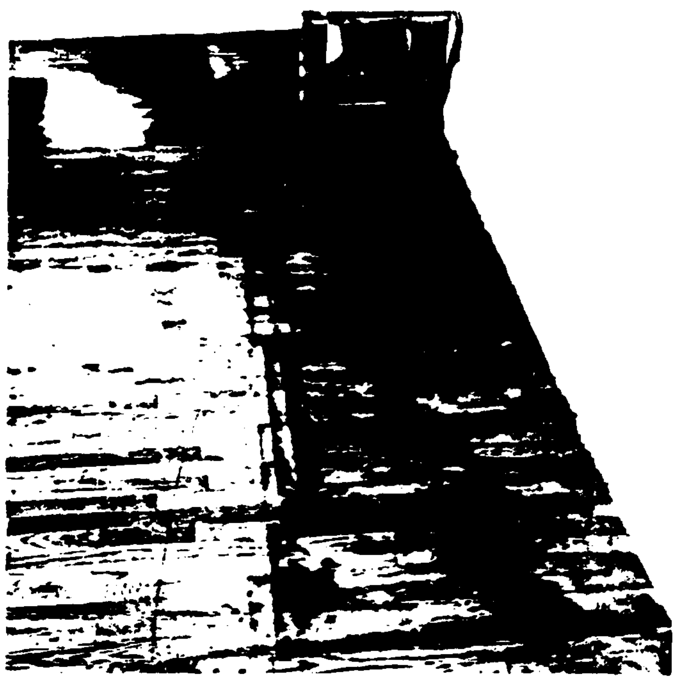

Figure 41. Stained floor in Room 219 where filing cabinets once stood.

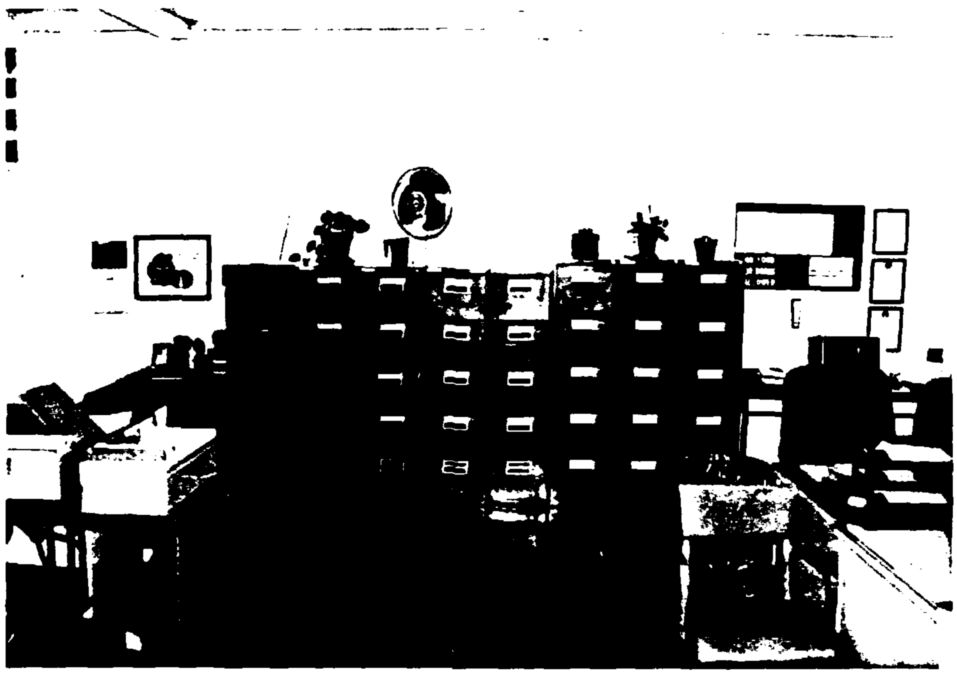

Figure 42. Filing cabinets along west wall of Room 219.

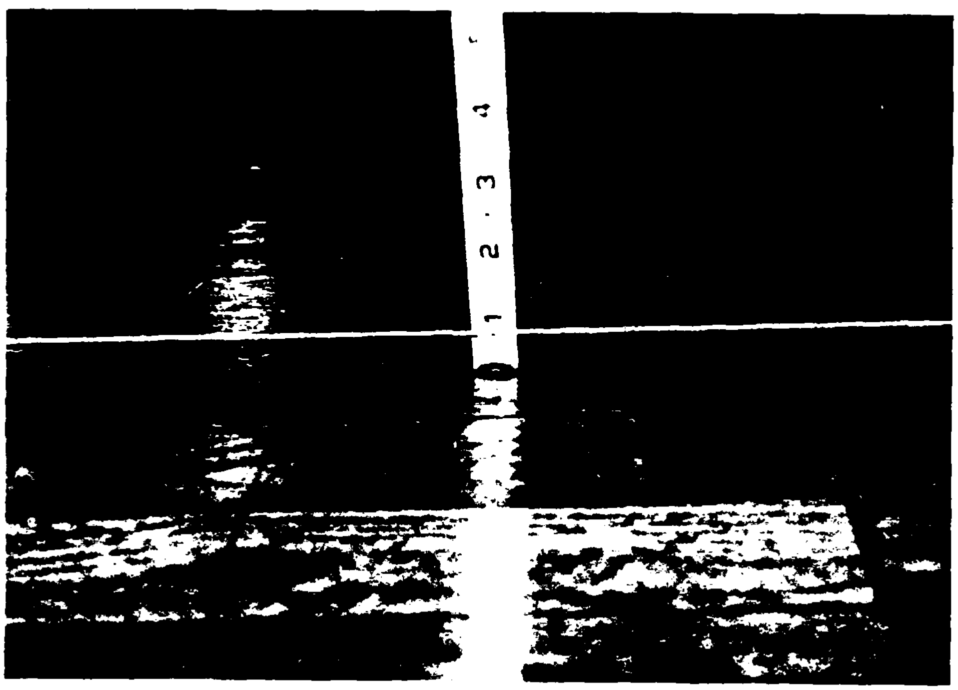

Figure 43. Deflection of floor in Room 210 between women's restroom walls below.

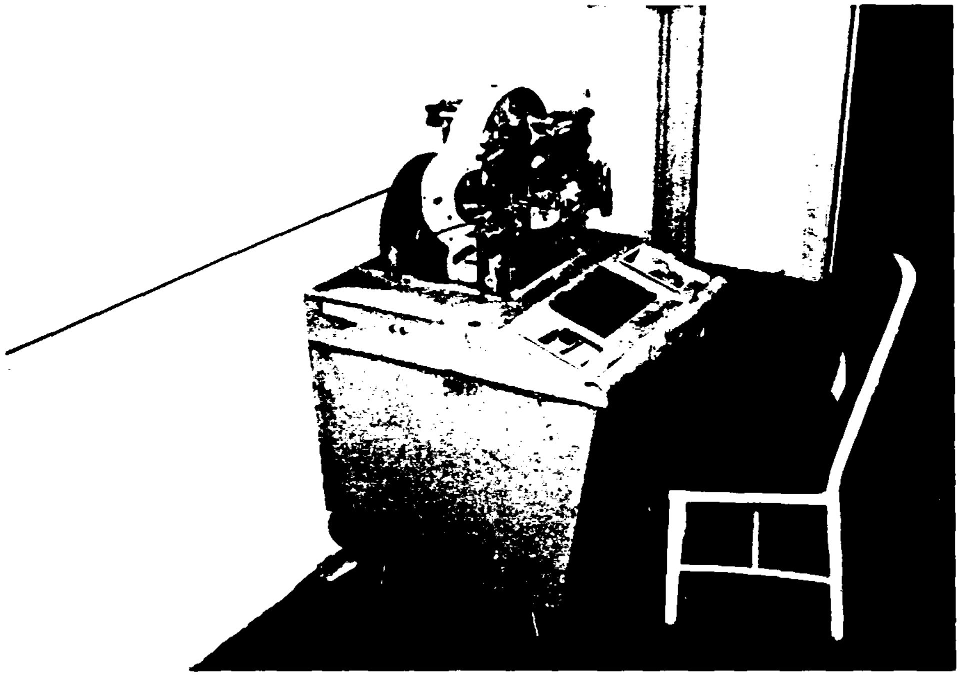

Figure 44. Stamping machine in Room 210.

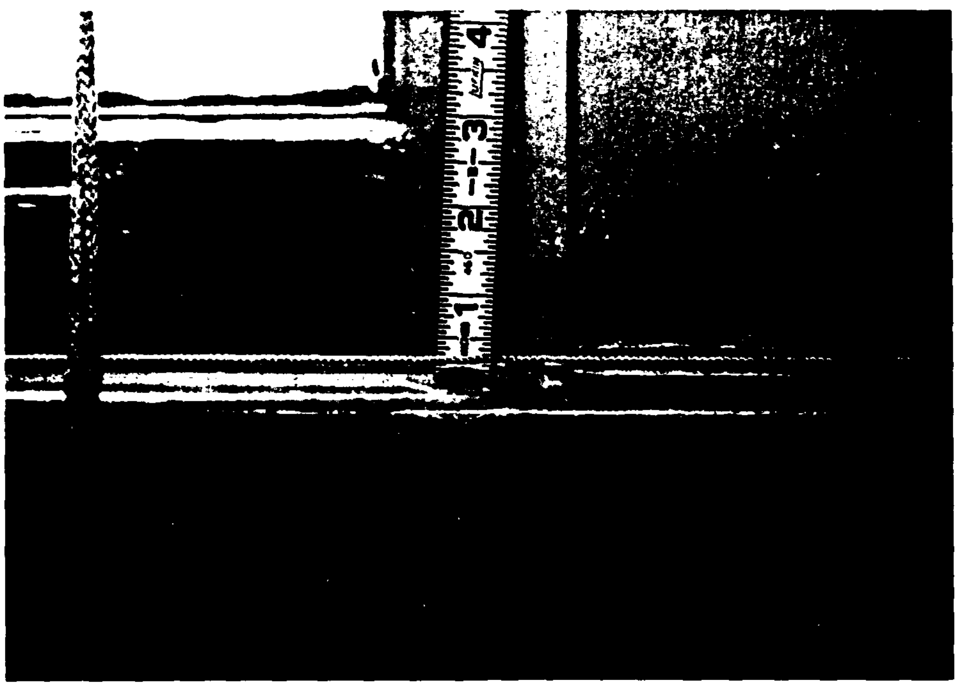

Figure 45. Deflection of window sill in Room 212.

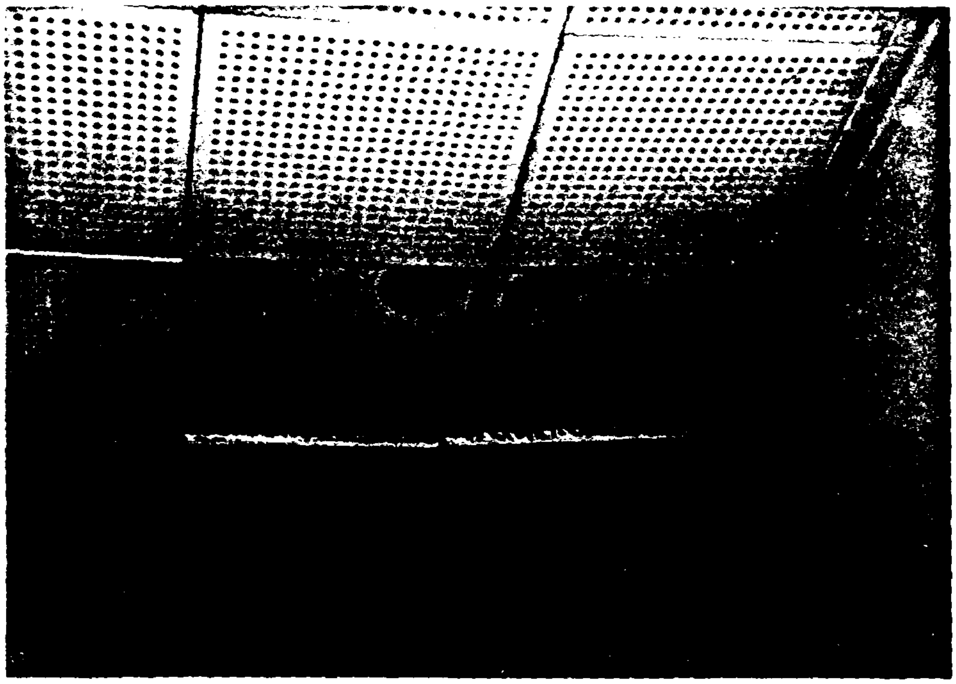

Figure 46. Ceiling crack in Room 104.

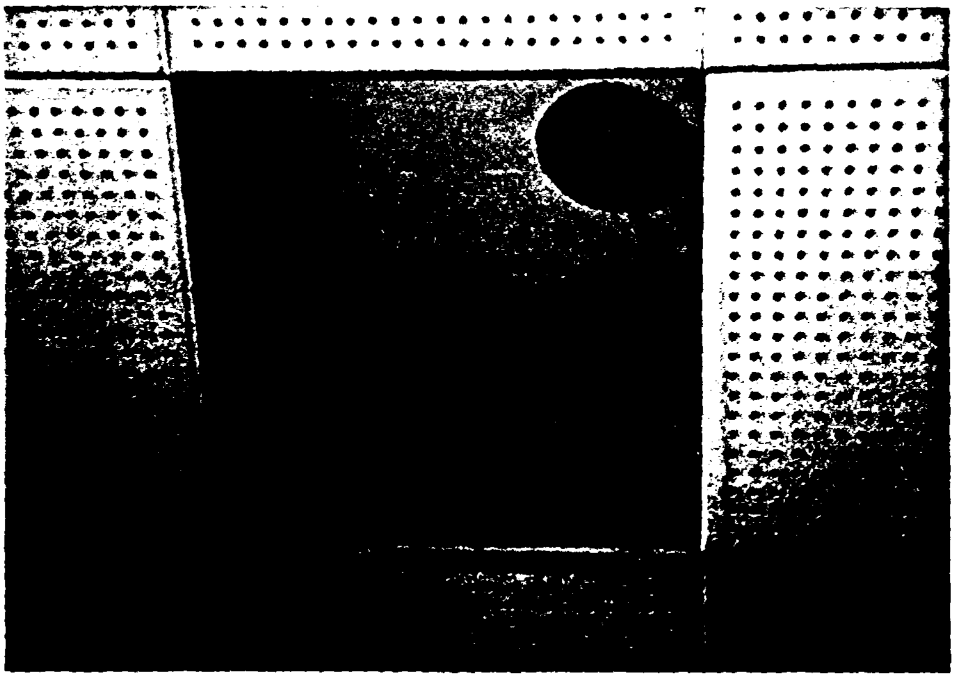

Figure 47. Ceiling crack in Room 114.

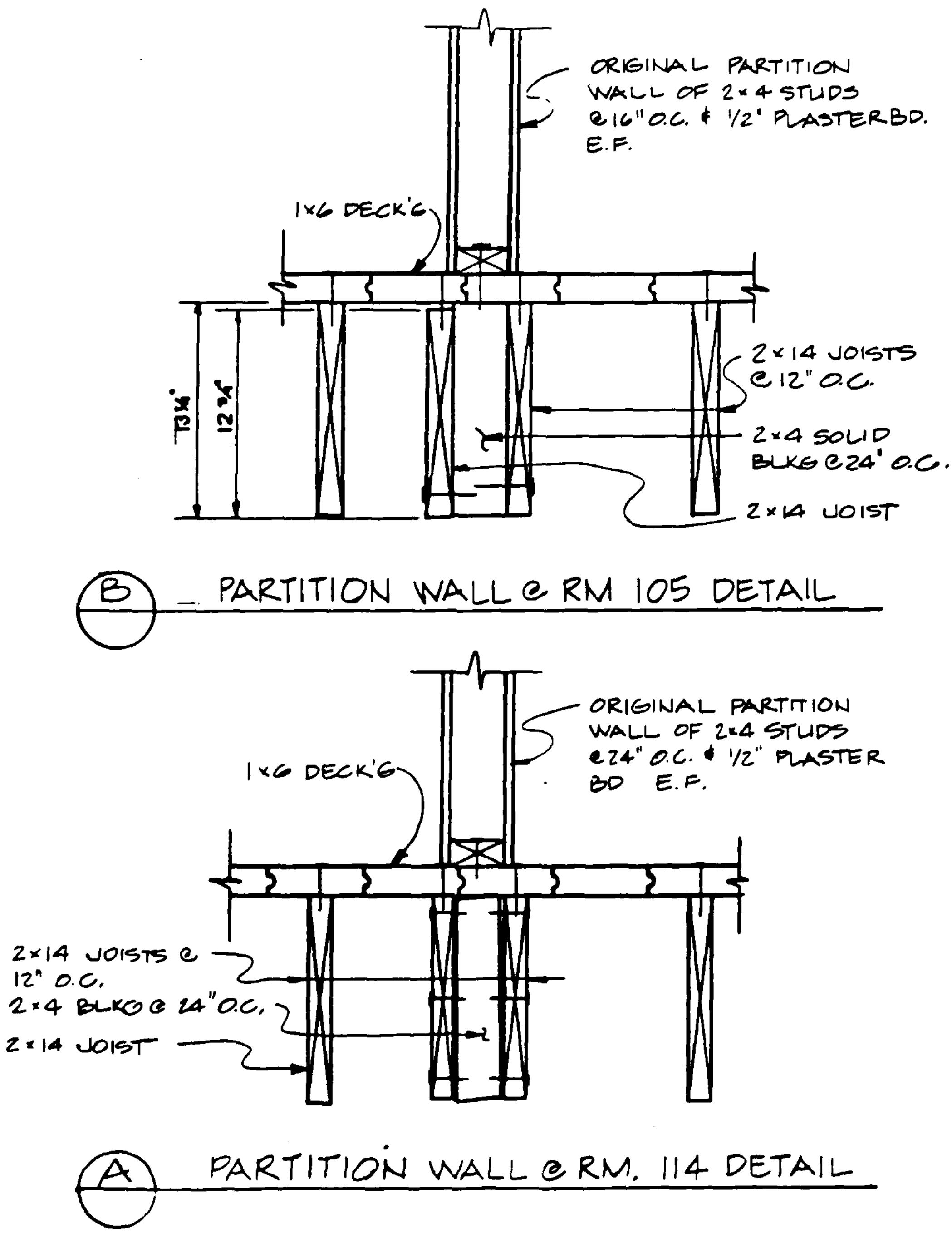

Figure 48. Partition wall details at base.

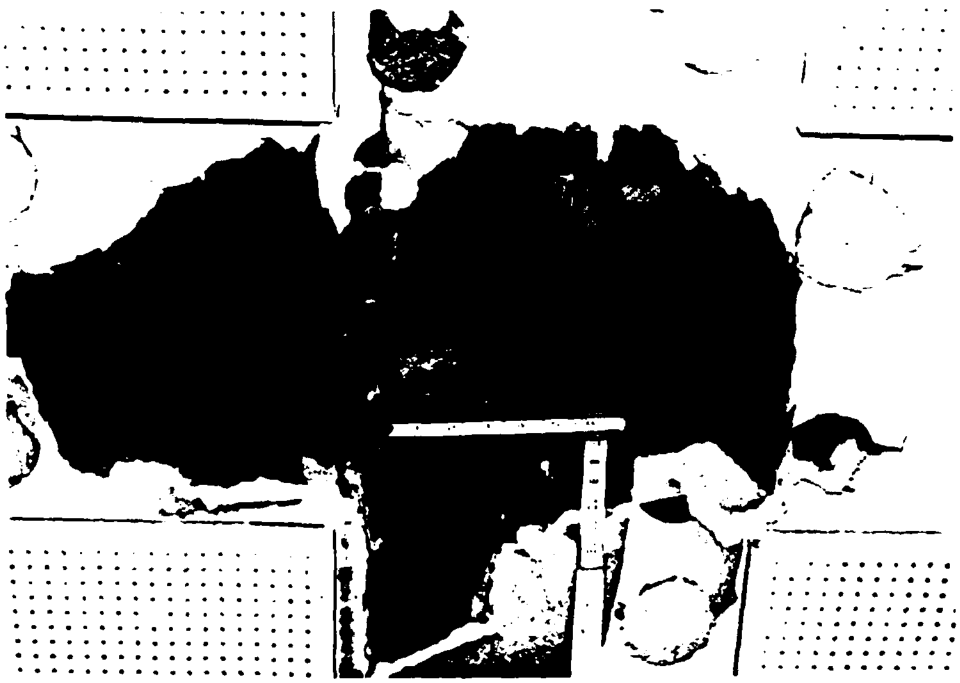

Figure 49. Condition of ceiling joist blocking in Room 114.

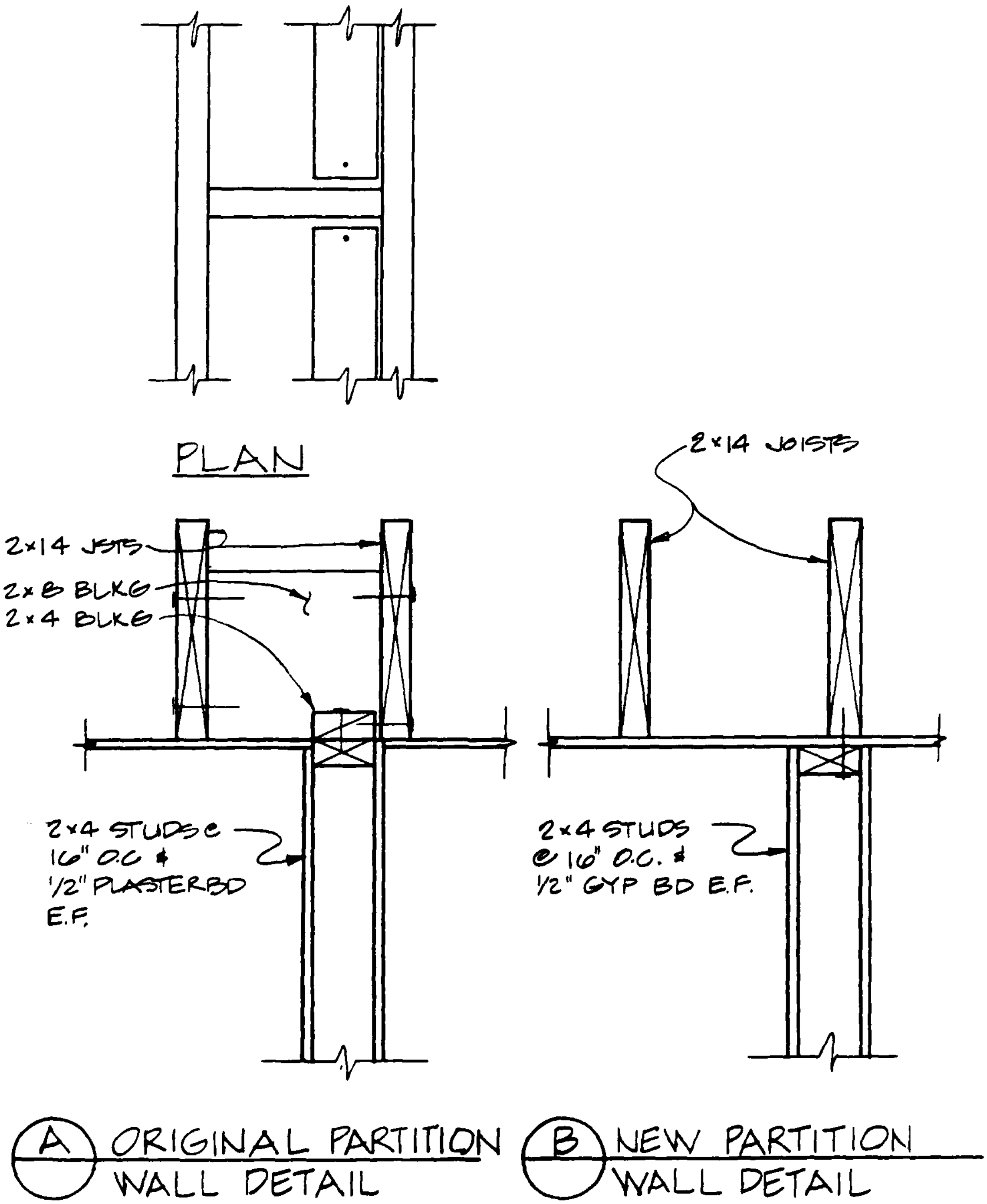

Figure 50. Partition wall details at top.

Table 3

Factors Used to Establish Allowable Bending Stresses

Duration of Load	Factor	Allowable f_b (psi)
Continuous	0.90	1305
2 Months (as for snow)	1.15	1668
7 Days (as for ponding)	1.25	1812
Wind or earthquake	1.33	1928

Table 4

Summary of 2 in. by 14 in. Roof Joist Analyses

Load Combination*	W_{max} (in.)	W_{tip} (in.)	M_{max} (ft-lb)	f_b (psi)
DL	0.90	-0.41	2013	572
DL + LL	1.30	-0.59	3773	1072
DL + SL	1.18	-0.53	3242	921
DL + W'	2.72	-1.28	3425	1305

*DL = dead load; LL = live load; SL = snow load; W' = ponding load.

Table 5

Summary of 2 in. by 14 in. Floor Joist Analyses

Load Combination*	W_{max} (in.)	M_{max} (ft-lb)	f_b (psi)
DL	0.23	1036	294
DL + CLL	0.86	3797	1078
DL + OLL	1.01	4487	1274
DL + RCLL	0.53	2347	667
DL + FDL	1.21	5660	1607

*DL = dead load; CLL = code classroom live load; OLL = code office live load; RCLL = realistic classroom live load; FDL = file cabinet dead load.

Table 6

Summary of 2 in. by 8 in. Floor Joist Analyses

Load Combination*	W_{Max} (in.)	M_{max} (ft lb)	f_b (psi)
DL	0.01	188	177
DL + LL	0.04	1438	1359

*DL = dead load; LL = live load.

Table 7

Summary of Second-Story Window Header Beam Analyses

Load Combination*	W_{max} (in.)	M_{max} (ft lb)	f_b (psi)		
DL	0.30	5776	1141		
DL + LL	0.56	10702	2114	>	1450
DL + SL	0.48	9305	1838	>	1668
DL + W'	0.61	11672	2305	>	1812

*DL = dead load; LL = live load; SL = snow load; W' = ponding load.

Table 8

Summary of First-Story Window Header Beam Analyses

Load Combination*	W_{max} (in.)	M_{max} (ft-lb)	f_b (psi)	
DL	0.14	4926	651	
DL + CLL	0.35	12362	1635	> 1450
DL + OLL	0.40	14220	1800	> 1450
DL + RCLL	0.24	8458	1118	
DL + FDL	0.18	7627	1008	

*DL = dead load; CLL = code classroom live load; OLL = code office live load; RCLL = realistic classroom live load; FDL = file cabinet dead load.

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